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Sustainability of Construction Materials

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Edited by J.M. Khatib

Introduction

1

J.M. Khatib
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Owing to the commending review received on the first edition of the Sustainability of Construction Materials' book (Khatib, 2009), we decided to produce a second, enhanced edition of the book. We added 14 chapters representing a wide range of materials and waste materials that can be used in construction. The original chapters were updated, except for Chapter 9 where it is thought that the 2009 version still valid and no major changes occurred.

As stated in the first edition, sustainable development is defined as 'a development that meets the needs of the present without compromising the ability of future generations to meet their own needs' (World Commission on Environment and Development, 1987). Sustainability is a broad term covering economic, social, and environmental issues. Sustainable development should be shaping the future of our planet and those living on it. Activities of human beings such as construction are having an impact on our environment. Many governments throughout the world have set targets to reduce the release of harmful gases (CO_2, SO_x, NO_x) into the atmosphere, as highlighted in the COP21 conference held near Paris in Dec. 2015. The construction industry consumes large amounts of raw materials. For example, in the United Kingdom alone, with a population of just over 65 million, the annual consumption of material resources amounts to more than 420 million tonnes, and large areas of land are converted from rural to urban areas (DEFRA, 2015). The extraction, processing and transportation of these resources emit high levels of carbon dioxide (CO_2) into the atmosphere, thus contributing to the pollution of the environment. The world consumption of these natural resources, especially by the construction industry, cannot be sustained at the present rate. Therefore construction professionals, including practising engineers, environmentalists, construction managers, researchers and academics all play a major role in sustaining our environment. This can be achieved through efficient utilisation of natural resources, reuse, and recycling of waste.

Many books on construction materials have been published. These books focus mainly on the engineering properties of such materials and little is devoted to environmental issues and sustainability. This book on sustainable construction materials aims to serve those professionals involved in construction in order to help them assist in achieving a sustainable environment. In addition to covering some fundamental properties of traditional construction materials that are used in construction, the book devotes sections to sustainability, including life-cycle assessment, embodied energy, and durability of construction materials. The construction materials examined in this book include aggregates (eg, natural and lightweight), concrete and cement replacement materials, geopolymers, masonry, timber, rammed earth, stones, bituminous materials, metals, glass, natural fibres, fibre composites, raw sewage sludge, gypsum, industrial by-products, desulphurised waste, wastepaper, and waste rubber.

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Before moving into the chapters concerned with individual construction material, Chapter 2 gives an overview of the principles of sustainability of construction and life-cycle analysis. After the general introduction that highlights the large consumption of resources because of construction activities and the need for adopting sustainable construction practises through the use of Life-Cycle Analysis (LCA), the chapter provides the general principles of sustainable construction. These principles include the general aspects of sustainability (eg, environmental, social, and economic), the various sustainability issues (eg, global warming, air and water pollution, acidification, deforestation, loss of habitat) and their connection to the construction industry. Also, the sustainable approach is discussed in terms of increased awareness, legislation and regulations and the demand for sustainable practises. The next part of Chapter 2 deals with the impact of sustainability on the selection of construction materials, which includes the wider impact of materials on various sustainability indicators, not only cost, availability and aesthetics. Other aspects are also covered, such as resource efficiency, energy and carbon, human and environmental health risks, support for social facets and well-being and support for sustainable processes. A detailed discussion on LCA follows. This discussion includes the general concept of LCA, its origin and associated standards, definitions and basics processes and generic concepts. The application of LCA in construction is detailed, including the three distinct levels for LCA evaluation, challenges in its application and the wider application of LCA in how construction materials are selected.

The physical properties that control the sustainability of construction materials are the subject of Chapter 3. These properties include porosity, pore size distribution and thermal conductivity. The different types of pores in cement-based pastes and mortar are explained as well as their pore size distribution. The diffusion coefficient of cementitious materials is described as it is linked to durability and thus the sustainability of construction materials. The coefficient is then correlated with accessible porosity affected by the water-to-cement ratio. Also, the correlation between porosity, pore size distribution and permeability is examined. The chapter goes on to describe the effect of porosity on heat transfer expressed in terms of thermal conductivity. The vapour–liquid interaction within a material is presented, including the ability of a material to absorb or release moisture. Towards the end of the chapter is a section on bio-based materials (eg, wood) with explanations on hygrothermal behaviour involving heat, air and moisture transfer.

Nanotechnology will play an important role in many areas, including construction. Therefore Chapter 4 focuses on the possibility of using nanotechnology in the production of sustainable construction materials. The chapter commences with a general introduction, a definition of nanotechnology, and recent advances in nanotechnology. Next, the possible general applications of nanotechnology in construction are covered, including titanium oxide (photocatalysis), carbon nanotubes, and the nanosilica. Owing to the small size of particles, there is a section on the possible negative effects of Si nanoparticles on health and the environment. Section 4.4 of the chapter provides examples of green nanoconstruction comprising the synthesis of nanosilica via a sustainable route, cement replacement with nanosilica, nanotechnology in alkali-activated materials, advanced construction materials using photocatalysis, phase change materials for energy storage, batteries and solar panels. The chapter ends with a section on future trends highlighting the need for further research and modelling, along with a proposal for new standards.

Chapter 5 focuses on the sustainability of glass in construction. The chapter starts with a general introduction stating the importance of glass as a structural material, followed by a description of silica glass and the production of soda–lime–silica flat glass sheets. The properties of glass are then discussed, including physical and optical properties, chemical and thermal properties, stress corrosion cracking and surface coatings. Other sections deal with the reduction that occurs in operational carbon when glass is used as a construction material, including the UK construction strategy. One section covers the features and benefits of using glass in buildings, which include daylighting, solar control, thermally insulated glazing and low-e glass, noise-controlling glass, vibration-reduction glass, self-cleaning glass and fire resistance glass. The use of glass in low energy/passive house buildings is briefly stated in Section 5.7. Section 5.8 describes various utilisations of glass as a construction material, such as the inherent energy and carbon of glass as compared to common construction materials, the sustainability of glass as a construction material and the recycling and reuse of glass. Section 5.9 covers the mechanical properties (eg, Young's modulus, strength) of glass, glass in load-bearing structural members (eg, toughened, heat-strengthened, laminated glass) and the failure mode and postfracture behaviour of glass. The next section discusses design standards, technical guidelines and recommendations for using glass in structural applications and connections in glass as structural members. The benefit of using finite element analyses and modelling in assessing the stress distribution is highlighted. The chapter concludes with a section on future trends.

Metals and alloys, which are often used in construction, are the subject of Chapter 6. The introductory section includes an overview of the chapter and talks about various features of metals and other aspects such as recycling and life-cycle assessment. The chapter comprises various sections covering ferrous alloys, stainless steel and nonferrous metals and alloys. The ferrous alloy section describes cast iron, wrought iron and steel. Included is a comprehensive description of the various types of stainless steel, such as ferritic, austenitic, martensitic, precipitation hardening and duplex stainless steel. Weathering steel is also described in a separate section. The nonferrous metals and alloy section depicts aluminium, copper and copper alloys and lead. There is also a section on weathering steel. Corrosion is related to durability, thus the various types of corrosions are described, including general, pitting, crevice, galvanic and high-temperature corrosion. Other aspects relating to sustainability and durability such as protective coating, design and selection of materials, cathodic protection, and corrosion inhibitors are described. Furthermore, towards the end of the chapter is a section on future trends and the need to prolong the life of components, with as little maintenance as possible.

The sustainability of timber and wood as construction materials is the subject of Chapter 7. It starts with an introduction to the importance of using wood as a renewable source in construction and other applications in order to reduce the emission of CO_2. The introduction also states that there should be a focus on a life-cycle assessment approach which covers all phases of the life of structures. The second part of the chapter deals with forest resources, the land covered by forest, deforestation, afforestation, illegal logging and forest certification. The chapter then goes on to describe the different forms of timber, such as round and sawn timber, engineered wood products

(EWPs), which are covered in Chapter 18, and wood composites. The thirst section of the chapter is concerned with the structural reliability of timber, which includes tree structure and growth, sawn timber, timber properties and moisture content. Next comes a section on the durability of wood covering decay such as biotic decay (fungal decay, insect attack) and abiotic decay (heat, oxygen, moisture, polluting elements, sunlight) of the wood. Preservatives and timber finishes against the different types of attack and weathering are also highlighted. Sections 18.5 and 18.6 are dedicated to life-cycle assessment, covering the LCA process, important considerations, function and functional units, allocation, system boundaries, carbon storage in the forest and wood products, embodied energy, carbon impact during construction, and operational phases and an end-of-life cycle (reuse, recycle, and energy recovery), as well as LCA case studies on completed buildings.

Dealing with the waste generated by the timber industry presents potential problems. For this reason, Chapter 8 focuses on sustainability of EWPs in construction and is different from Chapter 7 which deals with wood and timber. Chapter 8 deals mainly with adhesively bonded wood and timber that are made chiefly from waste in order to produce high-grade structural elements, thus contributing to the sustainability of our environment. The chapter starts with a general introduction, description of engineered wood products and the comparison of the mechanical performance of wood and sawn timber products. These topics are followed by a discussion about the environmental performance of EWP, which includes embodied energy, carbon and life-cycle assessment. In Section 8.4 of the chapter, the usability of wood fibre from harvested log is highlighted as well as the need for it to be utilised and recovered. Then come detailed descriptions about the applications for and manufacture of the various types of products, including finger-jointed timber, structural glulam, structural composite lumber, cross-laminated timber, structural I-beams, oriented strand board, plywood, chipboard and fibreboard. There is a dedicated section on adhesives that are used in EWP, including the service conditions, adhesive types, and wet bonding. In addition, other cross-laminated timber buildings are discussed, including the iconic 9 storey building in London, the design centre tower in British Columbia, and plans to build a 30 storey building in Vancouver, Canada and a 34 storey skyscraper in Stockholm.

Aggregates are the dominant materials used in construction. Therefore Chapter 9 considers the sustainability of aggregates in construction, along with the ways aggregates are produced, how they are extracted and processed, how they are transported and how they are reclaimed. The chapter also deals with their potential environmental impact and their mitigation, which includes changes to the landscape, the creation of noise and dust, vibrations from blasting, the impact on ground water and surface water, the impact caused by transportation and energy consumption. Best practises for managing the impacts are also included. This is followed by a discussion on the performance of aggregates now in use, substitutes and manufactured aggregates, waste products from aggregate mining and processing and how to extend aggregate availability through recycling. The sustainability of natural aggregates, which covers environmental, economic and societal values and responsibilities, are described. Life-cycle assessment of aggregate operations is explained as well as general approaches and issues related to the management of sustainable aggregate resources. Four case studies on the sustainability of aggregates from various parts of the world are included. The first case study focuses on government actions for resource protection and environmental restoration in Italy, while the second case study deals with government and conflict resolution in Canada. The third case study provides an example of corporate social responsibility for the expansion of a quarry, and the fourth case study highlights industry and transportation issues. The chapter ends with the future trends of aggregates in construction.

Chapter 10 deals with the sustainability of lightweight aggregates manufactured from waste clay. The earlier sections provide the background and the benefits of using lightweight aggregate in concrete applications and the added benefit if waste materials are incorporated into the process. The history of lightweight aggregates is the subject of another section. This discussion includes the development used by the Romans to construct the Coliseum (about AD 80) and the Pantheon (about AD 126). The various types of lightweight aggregates produced in the United Kingdom, their manufacture, properties and applications are described. The chapter moves on to explain the process of manufacturing lightweight aggregates from waste clay for structural and foundation concrete, which includes preparation of the clay and the kiln used for the production. The latter sections of the chapter are concerned with the environmental aspects and the CO_2 emitted to produce a certain volume of normal concrete as compared to the CO_2 produced using lightweight aggregates.

Chapter 11 gives an overview of masonry, mainly brickwork and concrete blockwork as a sustainable construction material. The chapter covers the manufacture of masonry units, including fired and unfired clay bricks, concrete blocks and mortars. The standards for masonry and its principal properties are covered (eg, compressive strength, density, configuration, movement, freeze/thaw resistance, active soluble salts, water absorption, fire resistance). The section on the historical use of masonry is followed by a detailed section on sustainability. It covers the basic definition of sustainability and masonry as a sustainable construction material. The next section focuses on quantifying the sustainability of masonry by using available techniques, including the Green Guide to Specification, the ENVEST software package, an Environmental Product Declaration, BREEAM and the Code for Sustainable Homes. Examples of other terms explained are the cradle-to-factory-gate, cradle-to-instal-onsite, and cradle-to-grave. Also, the chapter covers the masonry and the design life of buildings, the whole life costing, reclamation and recycling and the thermal mass of masonry. Examples of sustainable masonry construction are presented, including the BedZed building and the Winterton House in London, Queen Square in Leeds and the community centre in Swaffham, Norfolk.

The sustainability of natural stone as a construction material is the subject of Chapter 12. After a general introduction, the chapter describes the typical applications of stone in construction. The historic use of stone and stone resources in the United Kingdom and the extraction and processing of stone are described. The characteristics of different categories (sedimentary, igneous, metaphoric) of stone materials are the subject of Section 12.3. These include the durability of stones, moisture movement, mortar for stone and the repairability of stone structures. Next come the embodied energy and footprint of stones compared with other construction materials, whole life costing, and the thermal performance of stone-built structures. There is a section on the sustainable use of natural stone in construction, including the ability to reclaim masonry units, the use of stone in a modern context and the sociological sustainability of stone-built structures. The chapter concludes by indicating the future trends for the use of natural stone in construction.

The sustainability of compressed earth as a construction material is covered in Chapter 13. The chapter starts with a general introduction on the need to use materials for construction in a sustainable way. Then the chapter describes the environmental issues regarding the use of earth as a potential construction material. Compressed earth can suffer from exposure to rain, so there is a need to strengthen the materials by adding a stabilising material such as cement. However, cement requires high energy to manufacture, and reducing its utilisation is advantageous. In this chapter, a new technology to produce blocks using rammed earth is highlighted. This process is achieved through the use of shelled compressed earth blocks where a high-weathering resistance can be attained, as well as less use of cement as compared to rammed earth that is stabilised with cement and a sand-cement block. The social-cultural and economic issues are covered in this chapter. The chapter moves on to highlight sustainability as the focus of modern research, covering the advantages of earth constructions (low cost, sound and thermal insulation, energy saving, availability) and an embodied energy comparison with other building materials and the limitations. The various surface protection measures of rammed earth materials are covered in Section 13.7 (eg, cladding, facing, inlay, surface treatment, rendering, painting). The chapter moves on to describe the durability assessment parameters and a plausible sustainable option which is referred to as a ‘shelled compressed earth wall’. Both the production and the operation methods are described. Next comes a focus on the properties of the new product, including, net dry density, compressive strength, initial rate of water absorption, stress and strain, flexural strength and surface resistance.

The sustainability of bituminous materials is covered in Chapter 14, which is a new chapter in this edition. After a general introduction, the chapter describes the various forms of bituminous binders, including natural asphalt, refined bitumen and processed binder from renewable sources. The characteristics of bitumen and the various types of bituminous mixtures, including rheology, types, production methods, specifications and design guide are described. A section is dedicated to the sustainability by design, including performance and durability, reuse and recycling, retreading, repaving, ex situ recycling, ‘tar’ matter and recycling with foam bitumen and low temperature asphalts. This is followed by a section on preservative maintenance and repair, which includes preservative, rejuvenate and restorative treatments. The chapter concludes by suggesting ways for road construction with rammed earth in the future.

Concrete is consumed in large quantities during construction. Each human being consumes one tonne of concrete per year, which makes it second only to water as the highest consumed substance (Concrete Centre, 2015). Chapters 15–17 are dedicated to the sustainability of cement, concrete, and cement replacement materials in construction. Chapter 15 covers various aspects related to concrete, including life cycle, followed by a section on the raw materials required to make concrete. These raw materials include cement, supplementary cementitious materials, aggregates and admixtures.

The production of cement, the various types of blended cement, and the new clinker types are described. With regard to supplementary cementitious materials, the natural pozzolan, by-products, inert fillers, and manufactured products are described. Natural aggregate and recycled aggregate are covered. In the manufacturing of concrete section, various aspects of sustainability are covered. These include the reuse and recycling of concrete materials such as aggregates and water, the environmental impact and the use of self-compacting concrete, energy from plants, transportation and optimising concrete mix design. The various uses of concrete are highlighted in addition to demolition and recycling, including the CO_2 uptake. The chapter benefits from three case studies on sustainable construction. The first case study is on CO_2 uptake for a roof tile and an edge beam. A concrete bridge with various green solutions is the subject of the second case study, while the third case study focuses on the reduction of energy for heating and cooling. The future trends of concrete in construction are also covered. In addition, two more chapters (Chapters 16 and 17), as will be described later, address concrete materials. Chapter 16 deals with parameters that affect the durability of sustainable construction materials, mainly concrete and sometimes brick, which are not highlighted in Chapters 15 and 16. Various durability parameters are described in the chapter. These parameters include freeze/thaw, abrasion resistance, cracking, alkali–silica reaction, sulphate attack, chloride-induced corrosion and efflorescence. Cement is the most expensive and energy-intensive constituents of construction materials (eg, concrete); thus Chapter 17 is concerned with the production of cement-based materials with low clinker content. After the general introduction and the necessity of providing alternatives or partial substitution of cement using other cementitious/pozzolanic materials, the chapter describes the various cementitious or cement alternative materials and the chemical reactions involved. These materials are ground granulated blastfurnace slag (GGBS), natural pozzolan, fly ashes, silica fume, and metakaolin. The various properties of these materials as well as their effects on the performance of construction materials are covered. Properties include the origin/production of materials, compositions and physical properties, hydraulic properties, effect on concrete mechanical and durability performance. The mechanical performance section includes compressive strength, and the section on durability covers carbonation, chloride ingress, sulphate attack, and other deterioration mechanisms. The environmental benefits (eg, CO_2 emission) of using supplementary cementitious materials or low clinker cement are discussed. The chapter concludes with a section on future trends advocating the use of low clinker cement materials as a means of achieving sustainable construction materials.

The production of cement requires a high-energy output. Therefore Chapter 18 is concerned with the alkali-activated materials and geopolymers. Initially, the chapter describes the raw materials, activators and alkali-activated reactions. Then the fresh, physical, mechanical, and durability properties of alkali-activated materials are stated. Fresh properties include workability (consistence), working time and compaction. Mechanical properties cover strength, stiffness, shrinkage and creep whereas physical and durability properties comprise permeability, porosity, chloride ingress, carbonation, corrosion, freeze–thaw resistance, sulphate and acid attack, and fire resistance. The potential use of alkali-activated materials or geopolymers in structural applications is indicated, as well as the future trends of these materials in construction.

Chapter 19 focuses on the sustainability of vegetable fibres in construction. The beginning of the chapter provides general information related to the availability and extraction of fibre, the manufacturing and processing of raw materials, which include the various types of fibres (eg, sisal, coconut, bamboo, sugar cane bagasse, curaua, jute), and the advantages and disadvantages of using vegetable fibres. The general uses of the different types of fibres, including their use in cement and polymer-based composites as well as the environmental benefits of using vegetable fibres are included in the chapter. The chapter consists of two case studies based on the use of vegetable fibre in cement-based composites containing colloidal silica and in the production of particleboards using nonwood sources such as lignocellulosic biomass. The case studies include the raw materials required, preparation, testing methods, weathering conditions, mechanical and physical properties and the construction materials produced using vegetable fibres. The chapter demonstrates that using vegetable fibres plays a role in sustaining the environment, including social and economic aspects.

Chapter 20 deals with the sustainability of fibre-reinforced polymers (FRPs) as a construction material. After a general introduction and the definition of FRP, the chapter examines the use of FRP in the past, the present and the future as a material for construction, engineering and other applications. The different types, general properties and manufacturing process of FRPs are described, including polyesters, vinylesters, and epoxies. Then the chapter describes in some detail the use of FRP in civil engineering, building construction and transportation infrastructure (for structural and nonstructural applications, for strengthening and for external uses). This discussion is followed by a section on the durability of FRP, which covers moisture ingress, alkaline exposure, freeze–thaw, ultraviolet radiation, fire resistance and fatigue. Section 20.7 of the chapter is dedicated to sustainability of FRP materials. The section includes the life span of such materials (extraction and production of materials, manufacturing, use and reuse, and end of life), their embodied energy and a life-cycle cost analysis). There is a short section on the recycling of FRP. Towards the end of the chapter, the policies and standards for sustainable use of FRP are indicated. These include the Green Building Initiative in the United States and the Basic Work Requirement-7 (BWR-7) in the European Union for the regulation of construction products.

The sustainability of fibre composites in concrete applications is the subject of Chapter 21. The chapter begins with an overall view on the use of fibre since ancient Egyptian times and then moves on to focus on current practises in the use of fibre in concrete applications. The following section covers broader categories of fibre composites used in the building industry, including FRPs and fibre-reinforced cementitious materials and concrete. The chapter moves on to describe the fibres used in concrete. These fibres include organic fibres (natural fibres, which are described in detail in Chapter 20, and synthetic fibres), metallic fibres, mineral fibres, and glass fibres. The chapter then describes the properties of different fibres used (eg, aspect and modular ratios) and their effect on the performance of concrete, including stress/strain behaviour, ductility, crack control, and energy absorption capacity. The recent development in the use of fibre in concrete application is described. This topic covers the use of fibre in self-compacting concrete, hybrid fibre reinforcement and geopolymer concrete. One section describes the role of fibre reinforcement in achieving sustainable concrete.

Chapter 22 deals with the sustainability of wastepaper in construction applications. The chapter starts with a general introduction to the history of papers and moves on to the manufacture of modern papers and the generation of waste from the process. There are data on the large quantities of paper and paperboard produced by different countries. The need for paper recycling (including wastepaper sludge) is highlighted. The chapter next focuses on wastepaper sludge ash (WSA), including its production, particle size distribution, chemical composition, mineral composition and thermogravimetric analysis. Then comes a section discussing the properties of WSA and GGBS as a binder in producing construction materials (eg, mortar, concrete, compressed earth) with and without the use of cement. Properties include setting times, compressive strength and durability including sulphate resistance. The last section highlights the use WSA in the production of construction materials.

Chapter 23 covers the sustainability of waste rubber in concrete-based applications. After a general introduction on the amount of waste rubber produced, mainly water tyres, and the need for recycling and utilisation, the chapter describes the properties and classification of rubber aggregates. These include shredded, crumb, ground and slit (fibre) rubber. Then a section focuses on the fresh properties (eg, slump, air content, density) of concrete containing waste rubber. The effect of including waste rubber on the mechanical properties of concrete is also described. This section covers compressive strength, including the effect of interfacial zone of rubber and mortar, stress–strain characteristics (crack propagation, ductility), modulus of elasticity, toughness, impact resistance, splitting and flexural strength, load-deflection, abrasion resistance and bond strength. Another section deals with the physical properties of rubberised concrete, including water and capillary absorption, permeability, porosity, dry density, drying shrinkage, and thermal expansion. This section is followed by a discussion about the durability properties of concrete incorporating waste properties, including freeze–thaw resistance, chloride ion permeability, carbonation, fire resistance, effect of sea water, and acid attack. Towards the end of the chapter, the utilisation of waste rubber in other civil and construction applications is described.

Chapter 24 focuses on the sustainability of sewage sludge in construction applications. First comes a general introduction that indicates the need for effective utilisation and treatment of sewage sludge. Next is a section that describes the wastewater treatment processes, the chemical composition of raw sewage sludge as well as the different forms of sewage sludge. This section is followed by one on the management and production of sewage sludge and the utilisation of sewage sludge products in construction and civil engineering applications. Applications include ceramic and ceramic tiles manufacturing, lightweight construction materials, soil stabilisation as well as other applications (eg, absorbents, firing, clay alternatives). Then the chapter moves on to describe the use of cement and alternative binders to stabilise/solidify sewage sludge. The use of sewage sludge ash as a partial cement replacement material is also described, as well as the use of dewatered sewage sludge to make unfired brick. A new development on the utilisation of raw sewage sludge as water replacement in mortar and concrete is highlighted. The chapter demonstrates that there is a potential for the use of raw sewage sludge as water replacement in cement-based systems. The chapter ends by covering the environmental benefits and future trends of using sewage sludge in construction applications.

In recent years, there has been an interest in the use of gypsum as a sustainable mineral binder. Therefore, Chapter 25 is concerned with the utilisation and sustainability of gypsum-based construction materials. The chapter begins with a general introduction about the gypsum (composition, manufacturing, setting). Then the different types of gypsum products are described, including the global production of gypsum, the raw materials of natural and synthetic gypsum, chemical composition, dehydration and details on the manufacturing of β-hemihydrate, anhydrate, phosphogypsum, flue gas desulphurisation (FGD) gypsum and fluorgypsum. Some figures on the energy consumption and emission of gypsum binders are then presented. This section is followed by describing the reactions of hydration and the heat produced during reaction for different gypsum products and hardening. The mechanical properties (eg, compressive strength) and durability (eg, fire resistance) of gypsum-based binders are highlighted. The different products/composites made with gypsum (including waste gypsum) as binding materials are outlined (eg, masonry/concrete units) with more description of gypsum boards and panels, decorative elements as well as other products. One section is dedicated to sustainability aspects of gypsum-based products, including embodied energy and carbon footprint and reusing and recycling. The final two sections focus on life-cycle assessment and future trends.

Chapter 26 deals with the sustainability of desulphurised or FGD wastes coming from the coal power industry. After a general background on how FGD wastes are generated, the chapter describes the various desulphurisation processes (eg, dry, semidry, wet) and the types of FGD generated as well as the chemical composition of each of these types. The chapter describes the reactivity of the different FGD wastes when used in cement-based systems. Owing to the variable compositions of FGD wastes, the chapter examines the use of simulated FGD waste in order to determine the effect of chemical composition on the performance of these wastes in cement-based systems. Then the effect of FGD on the properties of concrete, when used as partial cement replacement, is examined. Properties include compressive strength, chemical shrinkage, porosity and pore size distribution and sulphate resistance. The possible application of FGD waste in the construction industry is highlighted, followed by the sustainability of FGD wastes in construction. The chapter ends with a section on future trends in which FGD wastes can potentially be utilised in a sustainable manner in various applications.

Because of the nature of this book and that different construction materials, such as brick, concrete, steel, and timber, are normally used in construction projects to produce, for example, a structure; thus a certain amount of duplication is bound to occur in the book (eg, see Chapters 15–17); however, these duplications were kept to a minimum. Also, because this book deals with sustainability, different authors used different approaches to sustainability, which should enhance the content of the book.

Finally, this book is a good reference of great benefits to all those professionals involved in the construction industry, including practising engineers, construction managers and associated professionals, environmentalists, policy makers, researchers and academics. Undergraduate and postgraduate students will find this book very useful. It is hoped that this book will increase awareness of more-efficient utilisation of natural resources and increase the use of waste in construction, thus contributing towards achieving sustainable development.

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Principles of sustainability and life-cycle analysis

2

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2.1 Introduction

Identifying the untenable consumption of the world's natural resources promoted strategic changes in resources management and highlighted a need for their effective and efficient use (WCED, 1987; Hill and Bowen, 1997). The construction industry is recognised as one of the largest consumers of natural resources (Kibert, 1994). The industry is responsible for the extraction of up to 60% of natural resources primarily used as materials for the construction of buildings and infrastructure (Hill and Bowen, 1997). It is also reported to contribute to almost half of carbon dioxide (CO_2) emissions, making it one of the principal contributors to global warming (Kibert, 1994). Governments, scientists and other stakeholders thus acknowledge the vital role the construction industry can play in a global quest for more responsible use of these resources. An extension to this acknowledgement is the proposition that construction activities should be managed in accordance with sustainable development objectives (Teo and Loosemore, 2003; Mustow, 2006). Thus, sustainable construction practices are now being prescribed for adoption in the construction industry. This chapter seeks to demonstrate how the life-cycle analysis (LCA also known as life-cycle assessment) could be used to progress the sustainable construction agenda, especially in the area of construction materials' selection, and to offer some guidelines for application. In the first part, the principles of sustainable construction are discussed. In the second part, the impact of sustainability on construction material selection and use is presented with a justification for the need for life-cycle considerations. In the final part, LCA origins, principles and application in construction material selection are discussed together with a review of contextual challenges and guidance for achieving easier and mainstream application.

2.2 The concept of sustainable construction

The provenance of sustainable construction is traceable to sustainable development. This concept is defined as development that 'meets the needs of the present generation without compromising the ability of future generations to meet their own needs' (WCED, 1987). It is broadly described as the judicious and equitable use of the world's natural resources without compromising the needs of future generations (Dickie and

Sustainability of Construction Materials. http://dx.doi.org/10.1016/B978-0-08-100370-1.00002-0

Howard, 2000). The underpinning of sustainability is based on three core principles identified by the World Summit on Social Development (UN, 2005). These are generally referred to as the pillars of sustainability, namely:

Environmental: Protection and restoration of natural resources, habitats and ecosystems.
Social: Ethical social responsibility and promotion of equality, well-being and social justice.
Economic: Equitable and fair distribution of economic resources.

The Earth Summit set out principles to be implemented according to an action plan (Agenda 21), requiring nations to develop strategies to achieve sustainability (UN, 1992). Subsequently, the Kyoto Protocol was agreed on under the United Nations Framework Convention on Climate Change. Collectively, these developments led to greater global commitment towards meeting sustainable development objectives. Specific actions have since been recommended, leading to increased legislation and regulation of sectors with the highest potential of contributing to the attainment of these objectives. The construction industry is one of these sectors due to its direct influence on heavy natural resource consumption as well as environmental and human impacts. As applied to the construction industry, the attainment of sustainability is achieving the right balance between the pillars (Hill and Bowen, 1997). According to Dickie and Howard (2000) and Zhao et al. (2012), this translates as ensuring the provision of current built environment needs without compromising resources needed to meet the needs of future generations. Some key sustainability concerns and their relationship with construction is presented (Table 2.1).

Table 2.1 **Selected sustainability issues and their linkage to the construction industry**

Sustainability issue	Connection to the construction industry
Global warming	Global warming is the general increase in global temperatures due to increases in the levels of carbon dioxide (CO_2) and other greenhouse gases (GHG). Total anthropogenic GHG emissions reached the highest levels in history between 2000 and 2010. This was estimated at 49 (±4.5) gigatonnes CO_2 equivalent per year in 2010 (see IPCC, 2014). The effects of global warming include extreme weather and natural disasters which threaten human existence. These emissions are mainly associated with industrialisation including construction activities. Significant GHG emission emanates from extraction, manufacturing, transporting, installing, use, maintenance and disposal of construction materials and products. Most of the embodied energy in construction materials is a result of CO_2 emitted from the use of fossil fuels for the generation of energy at different phases of the construction life cycle
Loss of biodiversity and natural habitats	Loss of biodiversity and habitat occurs as a result of clearance of land for construction or extraction of construction materials. This results in the loss of species and ecosystems or environmental quality that supports their existence

Table 2.1 Continued

Sustainability issue	Connection to the construction industry
Air pollution	Airborne particles (solid and liquid) and gases related to construction are often <10 μm in diameter, thus making them invisible. These often pose a risk to the environment and human health. Pollutants are emitted from construction and material extraction activities such as mining of aggregate, production of electricity, operation of equipment, manufacturing processes and transportation of materials/products
Acidification	Acidification occurs when gases like sulphur and nitrogen compounds dissolve in water or stream onto solid particles in surface waters and soils. This contributes to acid rains which affect ecosystems through a dry or wet deposition process. The primary sources of these acid rains are emissions of sulphur dioxide and nitrogen oxide from fossil fuel combustion. Activities that contribute to this include fossil fuel burning for the manufacturing and transportation of construction materials
Toxicity (ecological and human)	The emission of substances such as heavy metals can be poisonous to humans. Such emissions leave traces in the air and water which may affect human health especially when they reach intolerable levels. Activities that contribute to this include fossil fuel burning for the manufacturing and transportation of construction materials
Deforestation and arable land loss	Urbanisation is a leading cause of depleting forest resources and loss of arable land for food production. Forest and agricultural lands are cleared to make way for construction of buildings and infrastructure. Forest resources are similarly exploited for timber which is often used as a construction material. Less than 40% of the world's primary forests reserves remain but continue to be depleted to fuel urbanisation and related industrial activities such as construction. These forests contain more than half of the world's biodiversity and thus need to be maintained
Water resource depletion and pollution	Water resource depletion and pollution cause alterations in hydrological cycles, reducing the amount of water available for dilution of pollutants and human consumption. Construction activities and related extraction of natural resources require large amounts of water for processing. Associated pollutant emissions further pollute water resources. Impermeable surface of built infrastructure also reduces groundwater recharge

From Calkins, M., 2009. Materials for Sustainable Sites: a Complete Guide to the Evaluation, Selection, and Use of Sustainable Construction Materials. Wiley, Hoboken; Xing, Y., Malcolm, R., Horner, W., El-Haram, M.A., Bebbingto, J., 2009. A framework model for assessing sustainability impacts of urban development. Account. Forum 33, 209–224; Brabant P., 2010. A land degradation assessment and mapping method. A standard guideline proposal. Les dossiers thématiques du CSFD, No. 8, November 2010. CSFD/Agropolis International, Montpellier, 52 pp.; Bribian, I.Z., Capilla, A.V., Usón, A.A., 2011. Life cycle assessment of building materials: comparative analysis of energy and environmental impacts and evaluation of the eco-efficiency improvement potential. Build. Environ. 46, 1133–1140; IPCC, 2014. Climate change 2014: mitigation of climate change. Contribution of Working Group III to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change (IPCC). Cambridge University Press, Cambridge.

Various strategies promoting the mainstreaming of sustainable construction have emerged over the last two decades (see DETR, 2000; Dickie and Howard, 2000; Opoku and Ahmed, 2014). The principal recommendations for the construction industry to evolve towards sustainable practice include: increased profitability and competitiveness; ensuring user satisfaction and well-being; respect for people; protecting the natural environment; and reductions in the reliance on nonrenewable energy and natural resources (DETR, 2000). To achieve this, there is a need for a revolution in the manner in which construction is delivered and managed. The need for such sustainable approaches stem from the following factors:

Increased awareness: Global warming is now well acknowledged especially after global efforts towards reductions in greenhouse emissions since the agreement on the Kyoto Protocol (Hill and Bowen, 1997).

Legislation and regulation: Worldwide policies, notable within the EU and to some extent the United States, have promoted the incorporation of sustainable practice within the construction industry, a principal focus being the reduction of waste and recycling of material and responsible sourcing (Zhao et al., 2012). Such stewardship is increasingly required within the manufacturing industry, where most construction materials are made. There is an increasing urge within this sector to provide a closed-loop design process to encourage use of natural and recycled material (Calkins, 2009). These efforts also include the reengineering of products with new material compositions, including green chemistry to aid reduction of waste and pollution (Bribian et al., 2011).

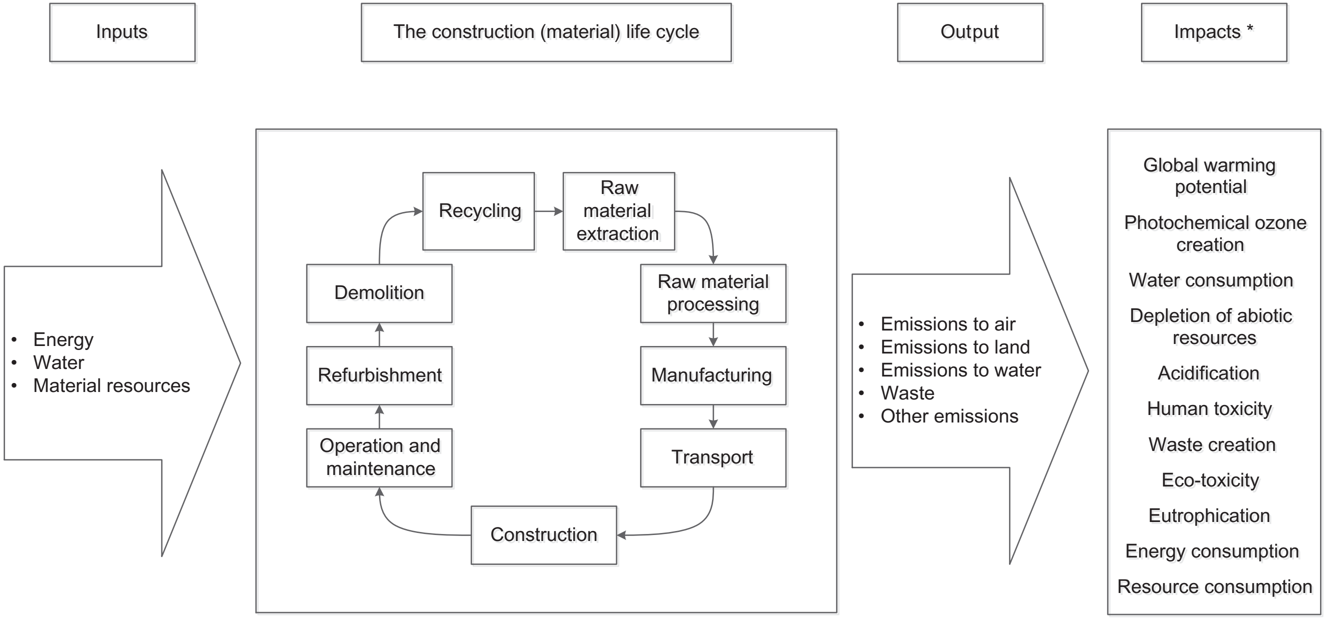
Demand for sustainable practices: The increasing awareness and legislation has led to increased demand for sustainable construction practices. This includes efforts by professional institutions to increase awareness and competence of members through the publications of policy documents and best practice guides. This has also led to an evolution of standards and criteria, particular schemes for assessing and certifying construction processes and products (Dickie and Howard, 2000). These schemes are primarily to ensure robust evaluation and accounting for the environmental and human impacts of construction products and services, as well as to support designers and specifiers in the selection of the most sustainable solutions among alternatives. Examples of the methodologies that have evolved include: environmental referencing methods rating alternative materials or substitutes; numerical computation of material impact on various sustainability indicators; and some specific methods for evaluating the embodied energy or carbon dioxide emissions as a single measure of impact (carbon accounting) (Dickie and Howard, 2000; Akadiri et al., 2013; Opoku and Ahmed, 2014).

2.3 Construction materials and sustainability

Construction materials constitute a major aspect of the delivery of buildings and infrastructure. A habitable square metre of living space within a building could require up to 2.3 ton of 100 different types of construction materials (Wadel, 2009). This includes the most commonly used materials (such as steel, iron, concrete, wood and glass) whose production is often within extractive and mining sectors (Calkins, 2009). Typically, the industry relies excessively on nonrenewable resources for construction materials, which contribute to the destruction of habitats and contamination of soil, air and water. Other attendant effects on society include poor health due to toxicity and the often deleterious effects from ecological imbalances (Dickie and Howard, 2000; Calkins, 2009). In the past, the critical factors that influenced the choice of construction materials were predominantly based on cost, availability and aesthetics. However, with the increasing emphasis on being sustainable it is important to consider the wider impacts of selected materials on various sustainability indicators. Some of the drivers of this requirement include an increasing need to attain the following specific objectives:

Resource efficiency: Materials with high reusability or recyclability to aid a reduction in continuous extraction of resources for new materials (Akadiri et al., 2013). This may also include a principle of reliance on more durable materials, which will last over the lifetime without a need for replacement. According to Calkins (2009), other factors that may aid the choice of resource-efficient materials include reprocessing potential, materials made from agricultural waste, renewable materials and procuring from agents with take-back policies.
Energy and carbon: Materials with low levels of embodied energy. This involves less reliance on materials that require fossil sources of energy for their manufacturing (Zhao et al., 2012). The procurement of materials locally could contribute to this through elimination of transportation costs (Akadiri et al., 2013). According to Calkins (2009), other factors that may aid in the choice of low-energy materials include: use of minimally processed materials; materials that rely on extraction or manufacture techniques with low energy use, water use or pollution; materials made with renewable energy (eg, wind and solar).
Human or environmental health risk: This describes materials that may emit toxins, pollutants or heavy metals into the atmosphere (eg, air, water or soil) (Jönsson, 2000). This includes materials with the least impact on construction worker health and safety as well as long-term user health (Hellweg et al., 2005).
Support social and general well-being: This refers to a need to select materials with cognisance to their ability to contribute to social and general well-being (Hellweg et al., 2005). Consideration must be given to the contribution of materials to economic or social sustainability including employment (Hill and Bowen, 1997). This also includes social corporate responsibility (BRE, 2009).
Support for sustainable processes: This refers to materials that support sustainable construction processes. Some materials may not be necessarily sustainable but support overall construction or usage operations with less net contribution to the environment (Calkins, 2009). This includes materials that require low energy or water consumption on site.

Guidelines and manuals for the attainment of the preceding objectives have emerged over the last two decades. These include green product standards, specifications, green procurement guidelines and sustainable design codes. Supporting these guidelines are schemes for assessments and certification such as BREEAM (United Kingdom), LEED (United States), Green Globes (United States and Canada) and several others, which provide criteria and standards for material selection. While these have provided organised process for making material selection decisions, a key limitation is usually a lack of appropriate consideration of the life-cycle performance and impact of these materials. Life-cycle considerations, however, require a strong methodological basis, especially techniques that consider the entire life cycle of a product or facility's existence (Ortiz et al., 2009). Fig. 2.1 depicts the typical construction life cycle, while Table 2.2 details some key sustainability concerns that arise at each phase. A sustainability concept that provides such methodological possibilities is the LCA technique. This is discussed in the next section.



**Most used impact categories in construction LCA studies (see Ortiz et al., 2009)*

Fig. 2.1 Life-cycle phases of construction, products and materials.

Table 2.2 Sustainability concerns at key life-cycle phases of construction, products and materials

Life-cycle phase	Sustainability concerns
Raw material extraction	Many construction materials originate from extractive industries. According to Calkins (2009), more than 3 billion metric tonnes of raw materials are used for manufacturing of construction materials annually. However, it is estimated that only 5% of material flows in developed countries are from renewable sources (Calkins, 2009). The main activities associated with the extraction of raw materials include mining, harvesting, clearing and dredging. These collectively cause destruction of ecosystems and habitats and generation of waste as well as excessive use of energy
Raw material processing	Processing of extracted materials requires large amounts of energy most of which is currently generated from nonrenewable sources. Due to high energy usage, the production of these materials is often associated with high levels of GHG emissions. For instance, the production of a kilogramme of aluminium can result in the emission of up to 15 kg of CO_2 (Gutowski, 2004). According to Calkins (2009), waste generation associated with processing could be as high as 3:1 (waste to metal ratio) for most metal production activities
Manufacturing	Secondary processing is often used to transform materials into various products through fabrication, assembly or finishing. As a result of more controlled industrial process, this phase involves less waste generation but may still involve substantial use of energy. High emissions to air and water remain, however, as well as human impact. This is due to the use of chemicals and solvents with intolerable levels of toxicity
Transport	All construction materials are often transported across the different phases of the construction life cycle. Freight activities can account for up to 30% of GHG emissions in the product life cycle. Overall, the transport sector contributed approximately 23% of total energy-related CO_2 emissions in 2010 (IPCC, 2014). Most modes of transport rely on vehicles and equipment that burn fossil fuels. Thus, distances, modes of transport and local availability of materials are critical factors in the determination of the sustainability of a construction material

Continued

Table 2.2 **Continued**

Life-cycle phase	**Sustainability concerns**
Construction	The construction phase poses its own unique challenges including waste generation and pollution from site activities. Workers are increasingly exposed to toxins and pollutants from construction materials and running of site equipment. Sustainable solutions include prefabrication of building elements offsite in controlled manufacturing environments. This, however, leads to potential use of heavier equipment on sites that may increase site energy consumption
Operation and maintenance	Over 80% of GHG emissions occur during the operation of facilities (Satori and Hestnes, 2007). In buildings, this is mainly attributed to the usage of energy for HVAC. In 2010 buildings accounted for 32% of total global energy use and 19% of energy-related GHG emissions (IPCC, 2014). This phase is the longest period of a facility's life, lasting periods as long as 50 years and over. Materials directly contribute to the energy performance of facilities due to their mechanical properties including insulation or thermal characteristics
Refurbishment	Most facilities need changes or enhancements in order to continue to be useful or functional. This requires reconstruction and retrofitting with similar sustainability issues as in the construction phase. When it is performed on a facility in use, it may expose users to additional human health impacts. Materials used during refurbishment or retrofitting could also play a key role in optimising a building's energy performance or sustainability during the operations phase
Demolition	Although this phase has the lowest associated energy consumption, most of the waste is generated at this phase. It is estimated that demolition could contribute as high as 2 tonnes of waste per m^2 of flooring of a facility (Lauritzen, 1994). This mainly consists of solid waste, hazardous substances and other pollutants
Recycling	There remains a high rate of use of nonrecyclable materials within construction. Construction waste could be as high as 40% of all waste generated with recycling of waste as low as 20–40% even in advanced economies (see DETR, 2000; Yuan, 2012). Many materials, however, have reclamation and recycling potential and thus could reduce the need for extraction of more raw materials. The use of recycled materials could save between 12% and 40% of total energy used for material production (Calkins, 2009)

2.4 The role of the LCA concept

LCA is an approach for evaluating the environmental impact of processes and products during their life cycle. LCA aids the systematic assessment of the environmental performance right from the extraction of raw materials through the manufacturing and usage phases and finally the end-of-life disposal or recycling (Bribian et al., 2011). Thus, LCA can be considered a whole life approach for evaluating the environmental impact of processes and products. Since the 1960s LCA methodologies have been used widely in other process-driven industries including manufacturing (Barnthouse et al., 1997).

2.4.1 Origins of LCA

The development of resource environmental profile analyses in the late 1960s is regarded as the genesis of modern LCA (Horne, 2009), an early example being Franklin Associates' investigation of the environmental profiles of different packaging alternatives for Coca Cola (Hunt et al., 1974). Between the 1970s and 1980s, multicriteria approaches for a systematic assessment of energy and environmental profile of products and services were beginning to grow within oil and manufacturing industries (Grant, 2009). By the 1990s, these processes became known as LCA after a proposition at a workshop in Vermont, United States, organised by the Society of Environmental Toxicology and Chemistry (SETAC). The North American and European SETAC LCA advisory groups met in Portugal in 1993 to produce a Code of Practice, which served as the main guidelines for LCA practice and was referred to as the LCA Bible (Fava, 2002). Many guidelines then proliferated. However, the most notable are the following series of ISO standards:

- ISO 14040 Environmental management, LCA, Principles and framework (1997/2006).
- ISO 14041 Environmental management, LCA, Goal definition and inventory analysis (1998).
- ISO 14042 Environmental management, LCA, Life-cycle impact assessment (2000).
- ISO 14043 Environmental management, LCA, Life-cycle interpretation (2000).

Beyond this period various concepts and approaches to LCA have evolved across various fields including building and construction sectors. This has led to the development of policy, standards and best practice guides for more systematic LCA approaches to assessing environmental impact (Barnthouse et al., 1997). Various extensions of the concepts have been proposed (Kotaji et al., 2002), while others have advocated simplification in order to make it more mainstream or widely applicable (Christiansen, 1997). LCA integration within management as well as further development of practices was promoted by the UN's LCA initiative in conjunction with SETAC (Horne, 2009).

2.4.2 Definitions and basic processes in LCA

LCA is defined as the compilation and evaluation of the inputs, outputs and the potential environmental impacts of a product system throughout its life cycle (ISO, 2006).

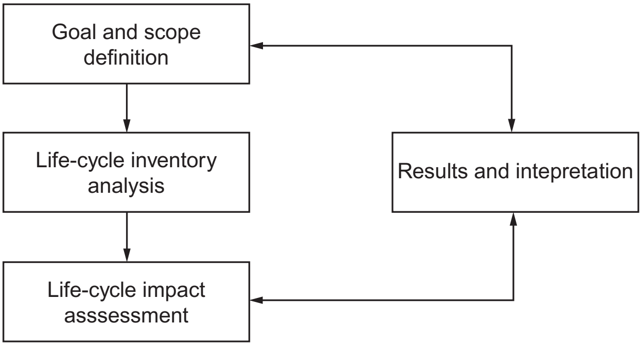


Fig. 2.2 Outline of generic life-cycle analysis (LCA) process (after ISO 14040). ISO 14040 (Environmental Management—Life Cycle Assessment—Principles and Framework). Permission to reproduce extracts from British Standards is granted by BSI Standards Limited (BSI). No other use of this material is permitted. British Standards can be obtained in PDF or hard copy formats from the BSI online shop: www.bsigroup.com/Shop.

The generic description of the LCA is based on the international standards organisation's series (ISO) 14040, which focuses on methodologies for LCA conduct. The ISO's LCA process primarily consists of four steps described below and depicted in Fig. 2.2.

> *Step 1 (goal and scope definition)*: The goal and scope definition establishes the functional units, system boundaries and quality criteria for inventory data. The goals must be focused on the specific subject being examined while the scope refers to the definition of system boundaries or coverage. Despite the traditional focus on environmental issues, this phase of an LCA should be used for establishment of sustainability objectives required in the application context. This can be done through broad stakeholder consultation and engagement.
>
> *Step 2 (life-cycle inventory)*: This phase allows the detailing of all inputs and output requirements of the LCA. The life-cycle inventory analysis deals with the collection and synthesis of information on physical material constituents and flows such as emissions or energy in various stages of the products' life cycle.
>
> *Step 3 (life-cycle impact assessment)*: The life-cycle inventory data is then synthesised through appropriate aggregation or assessment methodology. This aids the translation of flows into one or more impact categories such as global warming potential or human health impact. Characterisation factors are often used to calculate the contribution of each of the constituents for different environmental impact categories (reviews of LCA application in construction reveal the common use of the following impact categories which relate to the sustainability issues identified in Table 2.1: global warming potential; photochemical ozone creation; water consumption; water pollution potential, depletion abiotic resources; acidification; human toxicity; waste generation; eco-toxicity; eutrophication; energy consumption; nonrenewable resources consumption; air emissions; harm to human health and land use).
>
> *Step 4 (results and interpretation)*: This is the final stage where the summaries and analysis are reported. The communication should be transparent as well as clear about limitations, uncertainties and assumptions relied on in the assessment or parameter data used.

The principles of conventional LCA support the establishment of the environmental impact of products and processes. The impact categories can, however, be defined to include wider sustainability-related risk at the system level (Ortiz et al., 2009). This could aid analysts in considering multiple parts of a wider system as well multiple sustainability indicators (Bribian et al., 2011). Various related concepts and adaptations provide such possibilities, integrating social cost and cultural-related assessments (Kohler and Moffatt, 2003).

2.4.3 Generic concepts in LCA

LCA is not generally a single technique. It can be described as a concept consisting of general principles or basic requirements. These principles are geared towards the ability to determine the environmental impacts along a product or process life cycle. As a result, there is a plethora of approaches and scales for LCA. Variations are a result of the differences in the context of applications, including socioeconomic conditions or availability of resources or data (Horne, 2009). Notable differences thus exist in LCA standards and practices across different environmental, professional and industrial segments of society. Despite the plethora of approaches, there are basic elements that define LCA practice. According to Grant (2009), these are the use of functional units, system boundaries, exemption of input and output flows and assessment of impact on relevant sustainability impact categories.

Functional unit: The functional unit is the determination of a baseline or reference point to which all the inputs and outputs are measured. Examples of functional units used for LCA in construction include square metre (m^2) and cubic metre (m^3). The functional unit allows the standardisation of measures and inputs for all options that can be appraised in an LCA analysis (ISO, 2006). This aids comparisons of options based on the same benchmarks. In defining appropriate functional units, full consideration must be given to any secondary functions of the subject being studied (Kotaji et al., 2002). More expansive LCA methods now incorporate economic valuations as part of the units of assessment.

System boundary: Any unit subject of LCA analysis is often part of a system of interconnected elements. There must, therefore, be boundaries on which point to focus the analysis. The concept of system boundaries is usually framed around the life-cycle stages to be included for a specific analysis (Grant, 2009). This closely relates to the following concepts: cradle-to-grave, where full LCA is performed from resource extraction (*cradle*) through the use to the disposal phase (*grave*); cradle-to-gate, referring to *partial* product LCA from only resource extraction (*cradle*) to the factory gate, where usage stage is ignored; cradle-to-cradle, a specific kind of cradle-to-grave assessment, where the disposal phase is a recycling process; and gate-to-gate where a partial LCA may be used to evaluate only value-added processes in a production chain (Tingley and Davison, 2011). Boundaries should stipulate functional units, type and quality of data required by the LCA.

Inputs and outputs: LCA analysis is primarily based on evaluation of inputs and outputs required for a functional unit of production. The inputs and outputs normally include technical processes, materials and service flows (ISO, 2006). Elementary flows to and from the environment, such as minerals and land use, air, water, CO_2, nitrogen or heavy metals (Kotaji et al., 2002; Tingley and Davison, 2011). The aggregate flows are dependent on the type of LCA, the system being analysed or the impact areas being assessed (Grant, 2009). Typical inputs could be material quantities, equipment fuel or energy consumption utilisation rates. Outputs are typically in relation to sustainability impact categories (eg, energy, pollution, human health) in an appropriate measurement or functional unit (eg, kilojoule of energy).

Impact assessment: LCA always results in an interpretable outcome in terms of an impact to the environment (ISO, 1997). Despite variations in the type and number of indicators there should always be an indicator of the impact. Some methodologies prefer a life-cycle inventory phase in lieu of the impact assessment phase (Grant, 2009). This is, however, used when only minimal energy or emissions indicators are considered.

2.5 LCA application in construction

In response to the global quest for a responsible construction industry, LCA has gained mainstream focus over the last decade with early studies in the 1990s. LCA concept is increasingly recognised as a decision-support concept for the sustainable design, construction and operation of facilities (Forsberg and von Malmborg, 2004; BRE, 2009). This is primarily because of the benefits of applying a systematic and comprehensive method to optimise selection of product and processes with the least impact on environment and society at large (Cabeza et al., 2014). With the push towards sustainability, construction stakeholders are increasingly interested in incorporating LCA into construction decision making. Distinct forms of LCA practice have emerged, which are specifically tailored to meet some unique features of the construction sector (Ortiz et al., 2009). The environmental impact of construction often goes beyond simple aggregation of individual product assessment to a complex network of interdependencies. According to Ortiz et al. (2009), LCA within construction is either focused on specific products, components or the entire facility (building or infrastructure) and whole processes.

LCA applications for construction products, components or material: The main objective of using LCA in the selection of products, components or materials is to enable the elimination of less environmentally preferred options (Wadel, 2009; Bribian et al., 2011). The process involves individual evaluations, which seek to identify specific aspects of construction with the most significant environmental impact (Ortiz et al., 2009).

LCA applications for construction of whole systems or process evaluation: The simple accretion of individual component assessments does not provide a holistic view of the impacts of an entire facility or construction process. Consequently, several methods have been developed for assessing complete facility, system or construction processes. More comprehensive approaches for evaluation need to focus on the phases with the most environmental impact or opportunities for reducing impact (Cabeza et al., 2014). The focus and boundaries of an LCA must, however, reflect only the most critical components of the facility being assessed.

LCA tools and databases related to the construction industry: In order to meet the requirements and need for LCA, a number of tools have been developed specifically for construction. These tools provide a standardised approach to assessment and inventory data management. Various platforms have been used including web- or agent-based IT applications. In addition to the provision of a standardised methodology for assessment, some tools provide database capability for storage of both data and knowledge about various construction products and processes (IEA ECBCS, 2004; Bribian et al., 2009). These provide transparency between model outcomes and input data for evaluations (Cabeza et al., 2014). LCA tools can be leveraged through virtual digital technologies such as Building Information Modelling (BIM).

BIM-LCA integration will aid virtual prototyping and simulation of facility performance even before they are built (Anton and Diaz, 2014). This will save time and provide real-time option appraisal during the design of facilities.

The scope of these tools ranges from industrywide to product- or component-specific data. According to Trusty and Horst (2005), tools developed for construction LCA evaluation can be classified into three distinct levels or categories:

Level-1 product comparison tools: These are the types of LCA tools that focus on individual components or products used for construction. Prominent among these tools are databases where inventory or performance data for most material or products used within construction can be accessed. Notable developments in this area include: the BEES (Building for Environmental and Economic Sustainability) software by NIST (National Institute of Standards and Technology) (United States); National Renewable Energy Laboratory's (NREL) Life Cycle Inventory (LCI) Database (United States); SimaPro (United Kingdom); and Life Cycle Explorer application (Norris and Yost, 2001).

Level-2 whole-building decision-support tools: These are tools that have capabilities of evaluating the LCA or impacts of several components for a building. They include decision-support tools for optimisation of design or other broader phases of the construction process. Some developments in this area include: Athena (building environmental design tool) (Canada); and the Building Research Establishment's (BRE) Envest 2 (United Kingdom).

Level-3 whole-building assessment systems and frameworks: The final level refers to holistic frameworks' comprehensive assessment of facilities. These usually result in certification schemes that certify overall performance under which LCA is normally one of the critical determinants. Examples include: sustainable building certification schemes, such as BREEAM (Building Research Establishment Environmental Assessment Methodology) (United Kingdom) and LEED (Leadership in Energy and Environmental Design) (United States).

2.5.1 Contextual challenges in the application of LCA

Despite the advantages, many contextual construction issues pose challenges to implementing conventional LCA, which include

Lifespan of facilities: Buildings and infrastructure usually have very long lifespans, normally lasting over 50 years (Singh et al., 2011). Predictability of performance over such periods can be challenging, especially when changing their form or functionality.

Uncertainty and changes in use over lifespan: Retrofitting or refurbishment is commonplace for the long-term sustainability of facilities. Such activities may, however, result in significantly unpredictable changes in form and function. While flexibility to accommodate future changes is encouraged, it also exacerbates the predictability of performance of components or the facilities as a whole of the lifespan (Erlandsson and Borg, 2003).

Excessive contribution at usage stage: Most of the environmental impacts of facilities occur during its operation (Hill and Bowen, 1997; Jönsson, 2000). Such impact can, however, be notably reduced through appropriate design and selection of materials. For instance, it is estimated that this phase accounts for up to 90% of energy consumption, while material extraction or production accounts for between 20% and 30% (Satori and Hestnes, 2007). Other usage factors may, however, contribute further to impact, despite difficulty in dealing with such challenges during design. For instance, building occupant well-being and behaviour is not easily modelled within the remits of LCA, despite their excessive contribution to building performance (Singh et al., 2011).

Uniqueness of each facility: The level of minimal standardisation in the construction process is low. Thus, all new designs of facilities have peculiarities that make them different from others. There is constant need for new choices for any new facility. Several geographical and local environmental factors may affect construction. Since construction is site specific and almost always happens on a new location, several contextual or localised issues (such as microclimate) should be considered (Kohler and Moffatt, 2003). On a larger scale, regional climatic conditions affect constructions significantly. For instance, the energy requirements for HVAC (heating, ventilating and air conditioning) buildings in temperate regions are significantly different than in the tropics where HVAC demand is often lower.

Lack of integration: Design and production processes are often separate and organised around several different organisations. Thus, monitoring of the entire process can be complex. Furthermore, the design and construction decision makers do not produce/manufacture most of the components despite making key performance decisions (Ortiz et al., 2009).

Model complexity and standardisation: There is a need to rely on disparate LCA assessment for the multiplicity of products used for producing a single facility. Each material or product often has its own life cycle creating a web of complex interactions within the wider system. This may be even more complex to model because of the need for an interaction between these products and construction processes (Ortiz et al., 2009). The amalgamation of these disparate approaches and data poses a high degree of challenge due to its complexity (Erlandsson and Borg, 2003). However, there are significant efforts towards harmonisation and standardisation of LCA methodologies within construction. Geographical, national and professional diversity, however, affect effective cooperation for the attainment of this goal.

2.5.2 Towards the wider application of LCA in construction material selection practice

LCA is proposed as a viable approach that could aid material specification based on their sustainable credentials over the entire life cycle. In order to achieve optimisation of LCA in material selection practice, practical steps need to be implemented to ensure that the method comprehensively tackles all requirements for achieving a balance of sustainability. This includes synergistic integration with other concepts and methodologies for effective coverage of all pillars of sustainability beyond the traditional focus on environment. Practical approaches towards alleviating the contextual challenges will ensure further mainstreaming of LCA. This includes a need for appropriate goal and scope definition, which allows the evaluation of the requisite material performance attributes that can be incorporated into LCA with regard to both inputs and outputs. When integrated with concepts such as life-cycle costing (LCC), both the economic and environmental costs can be ascertained to aid the selection or specification of the best alternative materials to be used (Bribian et al., 2011; Ristimäki et al., 2013). The selection of appropriate materials is also dependent on regional or geographical differences, project budgets, as well as other functional and non-sustainability performance requirements (Cabeza et al., 2014).

In addressing challenges related to the multisystem complexities, Erlandsson and Borg (2003) proposed the use of a sequential and flexible structure instead of the

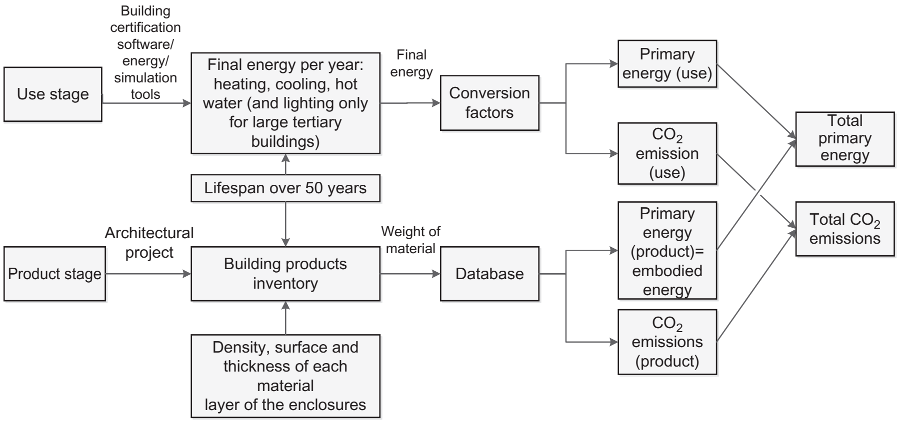


Fig. 2.3 Example of the structure of a simplified construction LCA methodology.
From Bribian, I.Z., Uson, A.A., Scarpellini, S., 2009. Life cycle assessment in buildings: state-of-the-art and simplified LCA methodology as a complement for building certification. Build. Environ. 44, 2510–2520.

usually linear and static approaches used in conventional LCA. Similarly, Collinge et al. (2013) introduced a concept of 'dynamic' LCA (DLCA). They defined DLCA as 'an approach to LCA which explicitly incorporates dynamic process modelling in the context of temporal and spatial variations in the surrounding industrial and environmental systems'. Fig. 2.3 is an example of simplified structure for LCA, proposed by Bribian et al. (2009), for the assessment of buildings in the usage stage. This model considered only two impact categories, CO_2 emissions and primary energy consumptions.

Huijbregts et al. (2003) developed a Monte Carlo-based simulation to mathematically quantify parameter uncertainty, which may sometimes be introduced by long life cycles, lack of data or imprecise LCA scope or boundary definitions. Other criticisms about the limitations of LCA application also include lack of consideration of some critical sustainability indicators, particularly those bordering on social principles, such as human health and well-being (Jönsson, 2000; Hellweg et al., 2005; Abeysundra et al., 2007). Some of these, however, fall within the remits of other evaluation methods with significant methodological differences that render their incorporation into LCA challenging. Several models are integrating other concepts, such as LCC, indoor climate assessments, cultural assessments and other construction-related health risk assessments (Jönsson. 2000; Kohler and Moffatt, 2003).

2.6 Conclusion

Despite the evolution in methodologies, notable differences still exist in the approaches adopted. The boundaries for most LCA application in the construction material sector have traditionally been based on an entire life cycle and typically over an average of

50 years. The findings and interpretation of existing LCA results reveal the urgent need for actions promoting newer techniques for the production of materials, use of local material, use of renewable natural resources, as well as recycling. Despite the success of LCA in construction, there remain challenges that need to be addressed. Data quality procedures need further development as a result of immature inventory data availability due to the novelty of the concept. However, this is not peculiar as even advanced applications of LCA suffer similar problems including incomplete inventory of some impact areas (eg, water use data). Another methodological challenge is the reliance on average systems and assumptions that often affects the granularity and level of detail required. The methodology requires constant updating in order to be able to reflect changes in policy, especially with the current drive for sustainability in industries across the globe. There remains a need for streamlined and contextualised adaptation of LCA. Such adaptations must take cognisance of impact categories with the greatest opportunities for optimising sustainable performance. Efforts towards standardisation must include wider stakeholder groups, particularly manufacturers and suppliers. These entities need to provide standardised information that can be synergised with emerging BIM technology. A recommended approach to achieving this is the mainstreaming of sustainable information declaration schemes for construction materials (eg, eco-labelling). Furthermore, LCA tool development must take cognisance of BIM data exchange requirements to ensure interoperability between LCA and BIM applications. There is a need for governmental and institutional promotion for the standardisation agenda in addition to development of frameworks that aid incorporation and application of policies and construction codes. In addition to the development of sustainable alternative materials, there is a need for optimisation of the impact of conventional materials. Undoubtedly, LCA provides such possibilities and needs to evolve with contextual consideration of challenges associated with its application in construction material selection, procurement and usage.

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Intrinsic properties controlling the sustainability of construction

3

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3.1 Introduction

Indoor air quality, healthy buildings, and material durability are important issues from both economic and social points of view. Construction materials, used externally or internally, can affect the health of building occupants as well as the energy consumption required while in service (eg, heating, hot water, air conditioning), thus its effect on the sustainability of the environment (International Energy Agency (IEA), 2007). Therefore, the preliminary selection of materials used for construction plays an important role in the success of the high environmental quality of building projects.

The normative rules (NF X 50-501, 1982) define durability as the ability of an entity to perform a function under given conditions of use and maintenance until the state limit is reached. More specifically, the durability of concrete is its ability to maintain its mechanical properties during its service life.

The durability is often concerned with the diffusion of harmful species into the building materials. For example, when talking about wood or reinforced concrete structures, the chloride penetration phenomena into concrete can't be ignored. Permeability is often associated with a material's durability as it relates to material diffusion. These two properties, diffusion and permeability, are interrelated; however, the question of their level of direct impact on material durability remains unclear.

In fact, very porous materials generally have pores at microscopic level, a complex and random pore structure where the role of heterogeneities becomes fundamental in the understanding of the physical phenomena involved and the prediction of behavior. Nevertheless, the heterogeneities related to the material morphology have not often been considered in the durability studies. In this context, the present chapter attempts to explore the link between the various physical properties of materials and their durability, while taking into consideration the material's morphological character, which is expressed in terms of porosity, as well as other phenomenon directly related to these properties.

3.2 Effect of physical properties on durability

A literature review on the different upscaling models (microscopic and macroscopic) of homogenization and the influence of materials and environmental parameters (temperature, relative humidity, pH, concentration, etc.) on durability are presented.

Sustainability of Construction Materials. http://dx.doi.org/10.1016/B978-0-08-100370-1.00003-2

Porosity is an important factor that reflects the morphological property of each material that influences the material's durability and consequently its sustainability. Therefore, parameters affecting the porosity are highlighted in this section. The 2D (3D) porosity is defined as the ratio of area (volume) of pores divided by the total area (volume) of the material. Theoretically, two types of porosity can be observed:

- Total or absolute porosity that refers to the ideal porous medium where all the pores are interconnected because it considers all the pores including those that are not invaded by the fluid (ie, closed pores).
- The effective or accessible porosity, defined as the ratio of connected pores, through which the flow takes place in the total volume of the medium. This definition is used only if the structure of the porous medium is known.

Fig. 3.1 shows the various types of pores or voids in materials (see Ollivier and Torrenti, 2008, Benboudjema, 2012 for more details). It should be noticed that the connected open porosity (ie, open interconnected voids) has a major impact on the diffusivity value of the material. Descriptions of the various void/pores in the text is useful as indicated in Fig. 3.1.

The porosity in the heterogeneous material (cement paste in this case) is formed through non-connected air bubbles, the capillary porosity, and also the hydrates porosity. The cement paste has porosity at different scales of observation. Therefore, its experimental characterization requires the use of different investigative techniques to the desired level. Although these techniques often require a sample preparation prior to test (eg, predrying in the case of mercury-intrusion porosimeter testing), they provide an estimation of the size of the pores and their distribution. In fact, the coarseness of porosity is generally underestimated due to exaggeration from the so-called ink-bottle effect, in which pores are assimilated to cylinders and artifacts related to the preparation of the sample. This may cause alteration to the pore structure due to the drying technique, the very high mercury injection pressure, and also the hysteresis of the intrusion and the release of mercury. There are generally two types of pores:

- Capillary porosity vestige of the initial clinker porosity where the pore size ranges from 0.01 to 50 μm. It seems that the structural differences between the cement paste, of various water to cement ratios (W/C), is mainly derived from the differences in the capillary pores (Bouny, 2007).

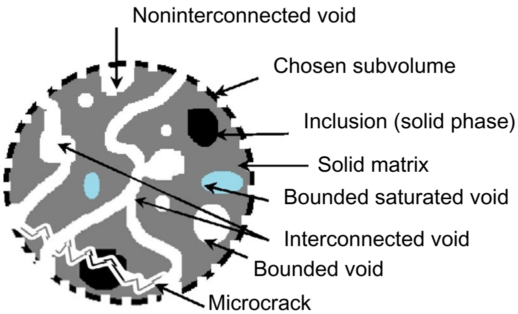

Fig. 3.1 Various porosities configurations.

- Gel C-S-H (calcium silicate hydrate) porosity which is the intrinsic porosity of hydrated products where the pores are small in size (ie, nanometer in size) where the dimensions range from a few angstroms to about 50–100 nm, which is much lower than that of the capillary porosity. These later can be further decomposed into interlamellar pores and intra-lamellaires cracks. This follows the C-S-H description adopted by various authors (Jennings, 2008; Nonat, 2004), but we can underline the limitation of continuous material approach on angstrom scale.

It is important to note that the C-S-H pores form a connected cluster, which is not generally the case for capillary pores, except for materials with very high W/C ratio. We have to underline that the mercury-porosimetry test shows tendency, but is unable to give quantifiable results at the necessary scale due to the high pressure used and the mechanical disorder that is caused.

The porosity of mortar and concrete is influenced by the evolution of microcracks during the early stages and the presence of aggregates. For compositions, W/C ratio of 0.4, containing silica fume and the mass ratio of the clinker by the silica fume is 10. The aggregates contact interface tends to increase the porosity to the surface of the grains through a destabilizing effect. On the other hand, portlandite appears preferentially in the vicinity of such an interface. This is obvious in Fig. 3.2A for materials without silica fume for different aggregate content (the W/C ratio is 0.4). For compositions containing silica fume (the weight clinker silica fume ratio is 10%) the coarse porosity (capillary) is higher when the aggregate content increases. This is less visible in the presence of silica fume (Garboczi and Bentz, 1992). The complex structure, time evolution, and the asymptotical porosity are related to the formulation and are predictable only for specific and chosen material.

3.3 Diffusion coefficient in cementitious materials

To ensure the sustainability of civil-engineering structures, knowledge of the properties related to diffusion is required. Diffusion refers to the transport process of a component in a given medium under the effect of its random agitation at the molecular level called Brownian motion (or vibrational on thermal aspect). If there are differences of concentration between two points in the medium, this phenomenon produces an overall transport component that is oriented from the most concentrated area to the least concentrated area. The diffusion process occurs even though the fluid (here water) doesn't move. Diffusion is a phenomenon that is irreversible in time. As it is difficult to follow the movement of each ion in a solution, the distribution is generally considered from a macroscopic point of view, that is to say that the overall particle motion is described by local concentration changes noted C. The ion will be described by its chemical potential ς by the relation:

$$\varsigma = \varsigma_0 + RT \ln C \tag{3.1}$$

where ς_0 is the chemical potential in a reference state, T is the absolute temperature, R is the gas constant and C is the concentration. The existence of a chemical potential

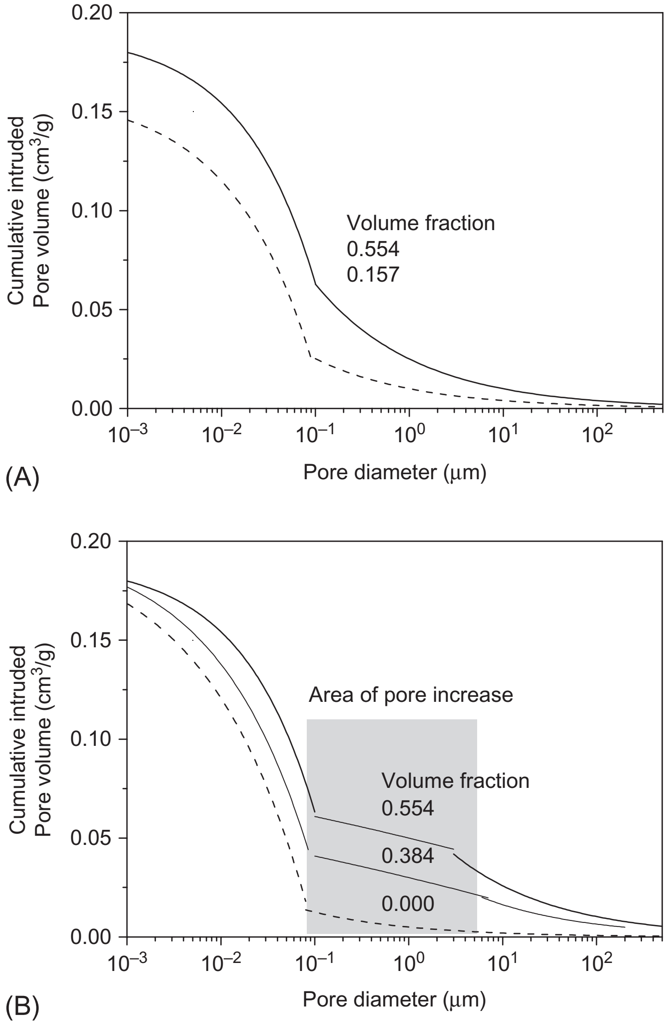

Fig. 3.2 Porosity distribution in pastes (A) and cement mortars (B) for different aggregate proportions.

gradient of a component *i* in a given medium causes a flow $\vec{J}$ of this component. The thermodynamics of this irreversible process postulates that there is a linear relationship between the flow and the chemical potential gradient acting as a thermodynamic force. Then we obtain the first Fick law, which gives the mass of solute per unit area and time[1]:

$$\vec{J} = -\rho D \vec{\nabla} C \tag{3.2}$$

---

[1] Could be written as $\vec{J} = -D\vec{\nabla}C$ where only the flux units change.

with D (m^2/s) being the diffusion coefficient of the species in the medium under consideration. The mass conservation in an infinitesimal volume element is expressed by the classical partial derivatives equation:

$$\frac{\partial C}{\partial t} = \mathrm{div}\left(D\overrightarrow{\mathrm{grad}}C\right) \tag{3.3}$$

In heterogeneous media, we can write $\boldsymbol{J}_\mathrm{p}$ ion flux passing through a defined unit area as follows:

$$\boldsymbol{J}_\mathrm{p} = -D_\mathrm{e} \cdot \vec{\nabla} C \tag{3.4}$$

where C is the average concentration on representative elementary volume (VER) and D_e is the effective diffusion coefficient (m^2/s). It is this factor that is accessed by the custom material without interaction between the diffusing species and the solid.

The ion flux through the material can be written by considering the material as a homogeneous medium or by considering the fact that the ions pass only through the porosity without integrating the complexity or tortuosity:

$$\boldsymbol{J}_\mathrm{p} = -D_\mathrm{e} \cdot \vec{\nabla} C = -D \cdot \phi \cdot \vec{\nabla} C \tag{3.5}$$

where

$$D_\mathrm{e} = \phi \cdot D \tag{3.6}$$

This shows that the effective diffusion coefficient D_e is lower than the clear fluid diffusivity (ie, diffusion coefficient D). So we keep in mind that in the case of pure fluid (at rest state), without electrostatic interaction between different ions, the diffusion through a liquid is related to the size of the spherical particles according to the Stokes–Einstein expression:

$$D = \frac{RT}{6\Pi N \mu a} \tag{3.7}$$

with μ being the water viscosity, a being the radius of the considered element and N being the Avogadro number. The principals' diffusivity values of different ions are given below (Table 3.1). As previously underlined and expected, the porosity has an important effect on the diffusivity value. Some authors (Benboudjema, 2012) collected

Table 3.1 Diffusion coefficient in pure water

Species	**Diffusivity ($m^2 s^{-1}$)**
HTO (tritium)	2.24×10^{-9}
Cl^-	2.03×10^{-9}
Ca^{2+}	1.56×10^{-9}

various results from the literature on effective diffusion-coefficient values (Fig. 3.3), whose principal product characteristics are given in Table 3.2.

Various changes in effective diffusivities as a function of porosity (accessible to water) on pastes and concrete based on cement and mineral additions are summarized in Fig. 3.3. It should be noted that the accessible porosity to water was

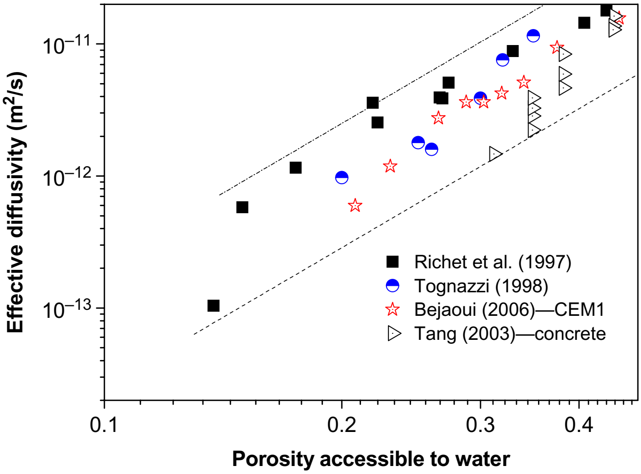

Fig. 3.3 Evolution of the effective diffusivity as a function of porosity for various cement pastes and concrete.
From Bogdan, M., 2015. Modélisation morphologique multi-échelles de matériaux cimentaires—application à la prédiction de propriétés effectives de diffusion (Thèse de doctorat de l'ENS de Cachan).

Table 3.2 Cement pastes and concrete diffusion coefficient

References	Cement type	Material type	Diffusing species
Richet et al. (1997)	CEM I	Cement paste	Tritiated water (T_2O)
Tits et al. (2003)	CEM I	Cement paste	
Bejaoui et al. (2006)	CEM I and V	Cement paste	
De Larrard (2010)	CEM I and fly ash (23% by mass)	Concrete	
Djerbia et al. (2008)	CEM I with silica fume (30% by mass)	Cement paste	Uranium
Tognazzi (1998)	CEM I	Cement paste	Leaching
Mejlhede Jensen et al. (1999)		Cement paste	Chlorides
Ngala et al. (1995)			
Tang (2003)		Concrete	
Carcassès et al. (2005)	CEM I, II, III, and IV		

From Bogdan, M., 2015. Modélisation morphologique multi-échelles de matériaux cimentaires—application à la prédiction de propriétés effectives de diffusion (Thèse de doctorat de l'ENS de Cachan).

calculated from the composition in the same case (Richet, 1992) tests and from the measurement of the mercury-porosimetry test. Under interactions between the chlorides and the cementitious matrix, the effective diffusion-coefficient evaluation was carried out using an isothermal interaction or by measuring the amount of flux through the sample.

Three mains findings can be extracted from Fig. 3.3:

- Except for chlorides, and therefore for tritiated water (T_2O) diffusivity, the use of mineral additives reduces the diffusivity dramatically by at least one order of magnitude.
- Generally, little difference exists between the cement paste and concrete, which may be due to the presence of the transition zone (aureole). Recent works minimize such assumptions (Bajja et al., 2015).
- In this case, the diffusion coefficient seems to be less dependent on the ions' nature.

Several results of the evolution of the effective chlorides diffusivity and various radionuclides of several results available from literature (the summary overview is described in Table 3.2) for different W/C ratio, different types of cement paste and concrete are grouped in Fig. 3.4. From these results the same conclusion obtained above can be confirmed, except that the effects of mineral admixtures seem even larger.

The effective diffusivity controls the diffusive process and the related material durability, but the weak level of diffusion amplifies the weight of transport phenomena induced by the flux over the material or flow over the porosity. The intensity of the flow and the ion diffusion amplification is related to the permeability.

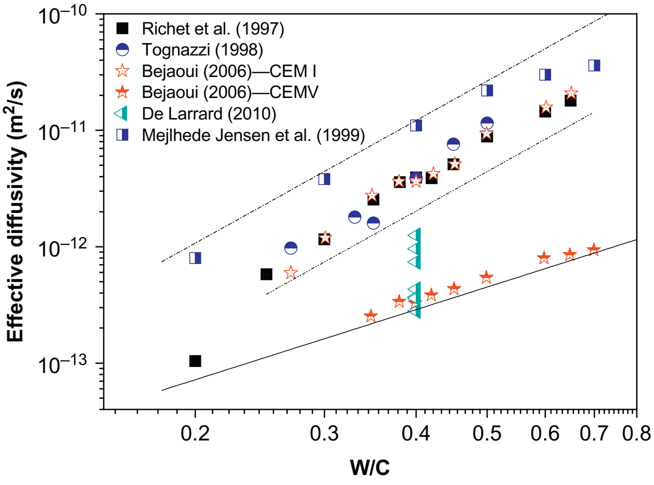

Fig. 3.4 Effective diffusivity evolution versus W/C ratio for different types of cement and concrete.

3.4 Correlation between porosity and permeability

A porous medium is primarily characterized by two interconnected macroscopic properties, which are porosity and permeability. Permeability is the fluid flow ability into a porous medium under the effect of a pressure gradient. The effective porosity gives an indication of the hydraulic permeability. However, the value of the permeability also depends on the pore structure, that is, interconnectivity and size distribution. The measured intrinsic permeability, in a given direction, is also relative to the open surface perpendicular to the fluid flow. Most porous materials have in fact a preferred orientation or an asymmetry of the porous phase in their structure, and the transmissibility of the saturated fluid does not happen in the same way in all directions. So as permeability or thermal conductivity change in direction, then the medium is anisotropic with respect to this property at the considered point.

A porous structure is considered heterogeneous if the property varies depending on the position of the point where it is located within the structure. Thus, in a porous material composed of several vertical isotropic porous layers, the properties such as permeability and thermal conductivity vary from one layer to the other, and constitute a generally heterogeneous porous medium. Furthermore, a porous medium is also heterogeneous because it is composed of a solid phase and a fluid phase (liquid/vapor/dry air). The porous medium is locally heterogeneous due to the large variation in characteristics that may exist within the solid matrix itself (eg, grains or fibers), as shown in Fig. 3.5. The homogenization technics allow for finding the condition to get globally homogeneous materials (on VER).

In addition to the high heterogeneity and anisotropy of most porous building materials, the latter contain localized cracks (Fig. 3.5B and C), which can influence the diffusion and especially the material permeability. Thus, there is a need to understand these phenomena by investigating the effective permeability, which is the principal parameter reflecting flow in porous media. Cracks represent preferential flow paths for the transport of fluid species and strongly contribute to the deterioration of structural

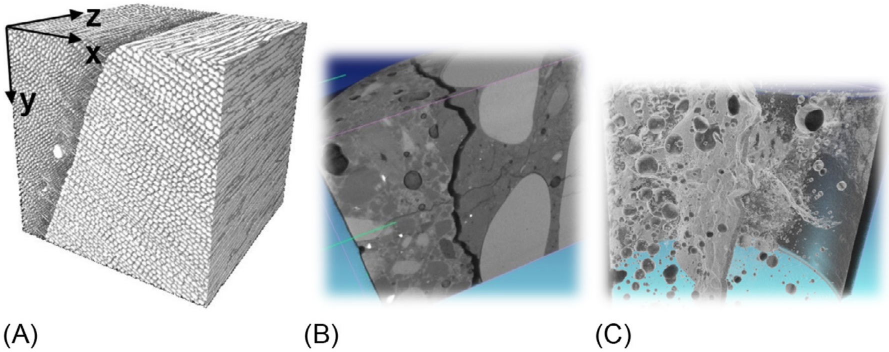

Fig. 3.5 Example of a locally heterogeneous and anisotropic porous media: (A) a 3D view of the spruce wood (B) and (C) cracked concrete with microbubbles.

performances by enhancing the apparent diffusive coefficient (eg, corrosion, carbonation, etc.) and safety (eg, radioactive fluid loss, radionuclide migration, etc.).

For example, in a recent study (Rastiello et al., 2015), investigation showed the effect of permeability on the fracture characteristic of porous materials where the mesoscale analyses of cracked porous volumes were performed. Mesostructures are defined according to the morphological framework proposed by Adler (2008), and recently developed by Roubinet et al. (2010). Several structures with different porosity are studied. They considered a cubical heterogeneous domain defined volumetrically by either a solid or fluid point defined as voxel information (Fig. 3.6). Each voxel represents an element of the solid matrix of the material and by consequence it represents the material morphology.

An explicit realization is given in Fig. 3.6 for different porosity from highly porous (0.7) to relative dense domain (0.05). This structure is obtained and characterized using three correlation lengths (6, 20, and 30) as shown from the granulometry illustrated in Fig. 3.7A (Bogdan, 2015).

The corresponding specific area (the interface between the solid structure and fluid subdomains) is shown in Fig. 3.7B where a maximum versus porosity is observed, which is due to percolation between dispersed solid grains and fluid channels. Such a specific-area is obtained through the analytical approach (Bogdan, 2015).

The porosity is also accessible analytically and checked numerically by integrating the solid and fluid phases associated with voxels information (see Fig 3.7B). Nevertheless in the statistical definition of the structure used it was found that the experimental values fit well with the porosity prediction. However, the computed minimum and maximum open sections (fluid plan in a given position) in comparison with the average section of the porosity domain (corresponding to global porosity) exhibit a constant difference for the different thresholds. Such deviation corresponds to the chosen variation and correlation length.

The relative variation of the porosity value increases as the threshold (or porosity) decreases (Fig. 3.7B). Based on known structure and geometrical information (eg, area, porosity, specific area), and by using the direct numerical simulation model, the equivalent thermal conductivity (or molecular diffusion) and permeability can be evaluated.

In this part, transport through macropores and cracks, as well as the diffusion in microporous solid inclusions, are taken into account. Uncracked porous volumes are considered first as a reference case. The pressure drop is normalized by the identified

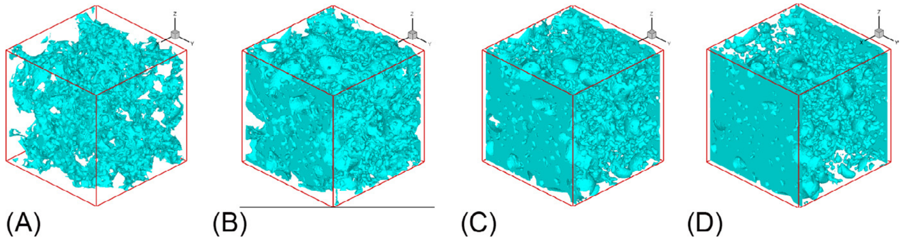

Fig. 3.6 3D field, from a strongly porous domain (A) reference case; to less porous: (B) 0.2; (C) 0.1; and (D) 0.05 (weak porosity).

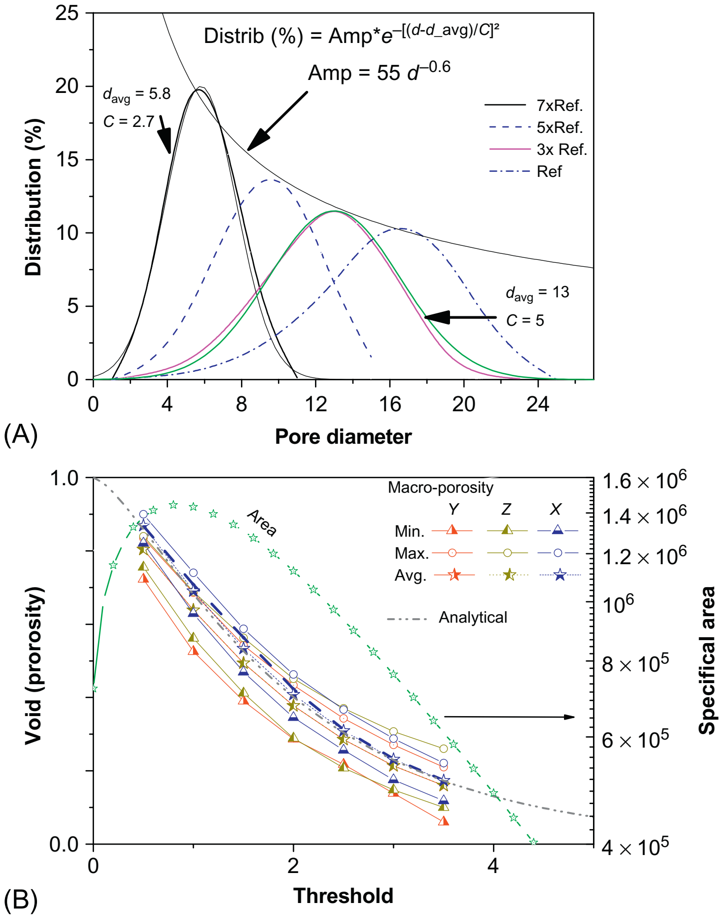

Fig. 3.7 Pore size distribution (A) and volumetric and surface porosity (B) versus the threshold level (complementary with Rastiello et al., 2015).

homogeneous permeability weighted by velocity term (V/Da), where Da is the Darcy number, a nondimensional parameter, representing the ratio of the permeability on the square passage section.

The cracks width effect on the average pressure gradient corresponding to two values of flow rates; 1 and 10 are illustrated in Fig. 3.8 for a porosity of 0.48. The pressure drop is normalized by the previously identified homogeneous permeability.

For microcracks the global flow doesn't change and remains horizontal and perpendicular to the section, and the pressure difference coincides with those obtained in the homogeneous case. For cracks with a width larger than 3% of the whole domain size (fluid and solid voxel ratio: 6/200), the apparent relative permeability changes. It can be observed that there is an excellent predictability of the pressure drop based on the previously identified permeability that is illustrated by the nonvisible effect of imposed flow rate (1 and 10). Such results can be expressed by an anisotropic tensor.

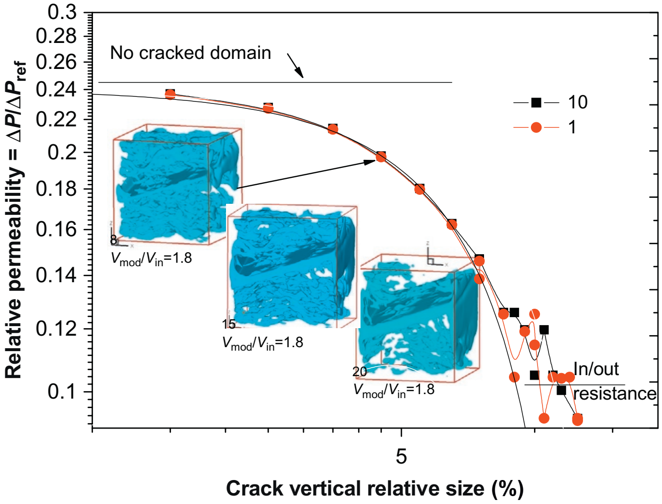

Fig. 3.8 Equivalent hydraulic permeability versus structure porosity (Rastiello et al., 2015).

When the crack width increases, the porous volume becomes more permeable as illustrated by the decrease in the pressure difference. For cracks having width larger than 15% of the global domain size, the pressure decrease exhibits asymptotical behavior resulting mainly from the entrance and exit of the fluid gaps and the crack channel. For such situations or intermediate crack widths a more complex phenomenon occurs with significant flow redistribution that affects the pressure-flow interlink. The flow change will induce a significant change in mass diffusion. In nonsaturated concrete, we face a more complex multiphase (liquid–gas) water distribution and time evolution. The complex phenomena evolve over time and are also interlinked with the temperature field.

3.5 Heat and mass interlinking

Heat transfer can also be appreciably affected by the morphology of the porous matrix and therefore its porosity, as mentioned above for fluid flow. Thermal conductivity is generally the most used parameter to describe heat transfer.

The models used to evaluate the effective thermal conductivity are very often based on the identification of the porous medium with a regular, simple geometric structure. Then, the conductivity is calculated by rigorous theoretical resolution, or by numerical calculation using simplifying assumptions. In the latter case, more formulations are obtained in explicit equivalent thermal conductivity (λ^*), which depends on the conductivity of the two solid–fluid phases and the porosity. The comparison of this training with experimental data is presented for fixed porosity in Fig. 3.9.

The existing expression remains inaccurate and unable to predict the equivalent thermal conductivity, the diffusivity, and the permeability. Several approaches

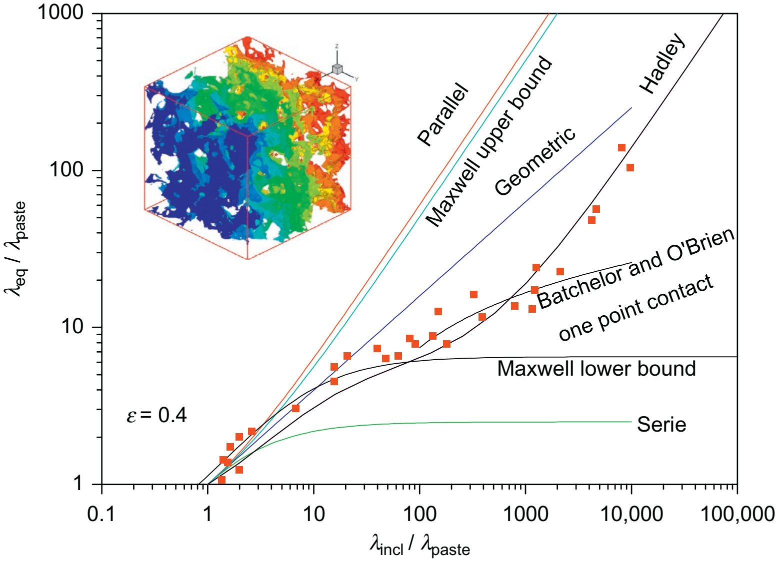

Fig. 3.9 Effective thermal conductivity for particle beds: theoretical prediction and experimental results (Kaviani, 1991).

including the porosity, tortuosity, or more complex structural parameters, allow some improvement with complex fitting. Based on the previously defined structure and known porosity, interface area (A_1), granulometry, and transverse surface, we will present a simplified scheme allowing for equivalence (for more detail see Rastiello et al., 2015).

The considered domain can be presented as three vertical multilayers where the central is a porous media of width ${}^{1}\!/_{2e_0}$ between two identical gaps of width e_0. The equivalent thermal scheme is series of three thermal resistances with the equivalent central as parallel thermal resistance composed by the fluid and solid phases. The geometrical ratio between the two parallel phases is given by the porosity ε. We insist on the interfacial fluid–solid area (A_1) controlling the coupling between the solid and fluid phase and the tortuosity. The equivalent, simplified resistance is given according to the following equation:

$$\frac{\lambda_f}{\lambda_{eq}} = 2e_0\left(1+\frac{\left({}^{1}\!/_{2e_0}-1\right)}{\varepsilon+\left(1-\varepsilon\right)\lambda_r}\right) \tag{3.8}$$

with

$$\text{When}\quad \lambda_r \rightarrow 0: \quad \frac{\lambda_f}{\lambda_{eq}} \rightarrow 2e_0\left(1+\frac{\left({}^{1}\!/_{2e_0}-1\right)}{\varepsilon}\right) \tag{3.9}$$

and

$$\text{When } \lambda_r \rightarrow \infty : \quad \frac{\lambda_{eq}}{\lambda_f} \rightarrow \frac{1}{2e_0} \tag{3.10}$$

where $\lambda_r = \lambda_s / \lambda_f$. The analytical apparent conductivity is given by $\lambda_{eq}^r = \dfrac{\lambda_{eq}}{\lambda_f}$.

The obtained effective conductivity ratio for the different thresholds and solid–fluid conductivity ratio are shown in Fig. 3.10. The apparent conductivity increases with the increase in solid–fluid conductivity ratio. For high porosity ($\varepsilon = 0.7$, ie, threshold 3), the apparent thermal conductivity change with a ratio of two when the relative conductivity changes by the order magnitude of 10^2. For low porosity, the change is more pronounced. The obtained numerical results do not fit with the analytical part and do not tend to with the two-limit cases, represented on lower part of Fig. 3.10. This is illustrating the more complex phenomenon that is resumed by either the important part of the interfacial surface exchange (specific area) allowing the thermal coupling between the two considered phases in the intermediate transitional domain and the tortuosity of the solid or the fluid phases on the two asymptotical cases.

$$\lambda_r \rightarrow 0 \quad \frac{\lambda_f}{\lambda_{eq}} \rightarrow 2e_0 \underbrace{\left(1 + \frac{\left(\frac{1}{2e_0} - 1\right)}{\varepsilon}\right)}_{\text{Corrected by tortuosity}} \tag{3.11}$$

$$\lambda_r \rightarrow \infty \quad \frac{\lambda_{eq}}{\lambda_f} \rightarrow \frac{1}{2e_0} \tag{3.12}$$

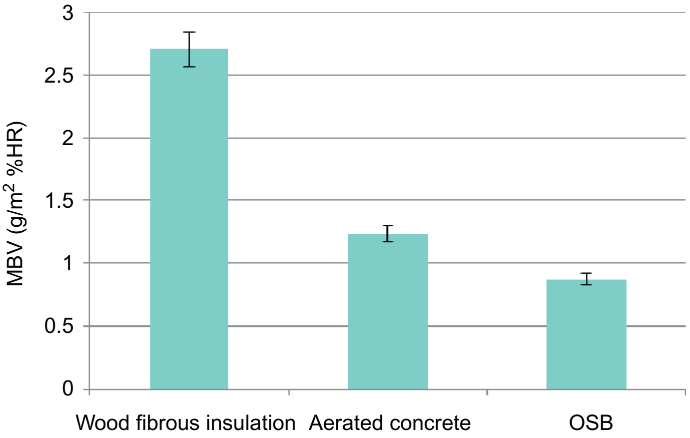

Fig. 3.10 Moisture buffer value of wood fiber insulation, cellular concrete, and OSB obtained after three cycles of wetting–drying.

Therefore, the ability to generate a controlled porous structure with known porosity and specific area is demonstrated. The porosity evolution modifies the apparent granulometry, the specific area, and the tortuosity. Moreover, in multiphase fluid (Bories et al. (2008)), the adsorption and capillary pressure modify local constraint and affect the previous underlined geometrical characteristic.

3.6 Vapor–liquid interaction

This section is concerned with the vapor–liquid interaction inside a material. Such interaction is related to the previous diffusion and permeability but could illustrate strong correlation with the physical and chemical adsorption phenomena. Physical adsorption involves the condensation of molecules on the surface of the solid material. Adsorbed molecules are weakly linked to the solid by intermolecular forces of attraction (mainly van der Waal's forces). The energy involved is <20 kJ/mol. There are no privileged sites and adsorption can be multimolecular, that is, that a second layer, then a third, can come to adsorb on the first. It is a reversible phenomenon since the adsorption is little influenced by the nature of the surface.

Chemical adsorption is due to ionic or covalent chemical bonds between gas and solid surface. This implies a change in its structure or reactivity. The energies involved are more important. They vary from 80 to 400 kJ/mol. Chemical adsorption can be monolayer. It is an irreversible phenomenon that is specific because it depends on the nature of the chemical bonds that the atoms in contact may contract.

Such adsorption and desorption processes could be seen as the effect of the environment on the materials, but due to the water contain reversibility, it could be analyzed as a material contribution in regulating the ambient building vicinity.

The material contribution in regulating the ambiance vicinity can be directly reflected by the hydric and morphological parameters more specifically the specific surface. This parameter, which is inversely proportional to the length, plays an important role in the adsorption problems. As for porosity, sometimes it is necessary to distinguish the accessible surface area and the total surface area including the area of the walls of the pore surfaces. This parameter is strongly dependent on the porosity as shown in Table 3.3.

A material's ability to absorb or release moisture is sometimes called the moisture buffer capacity of the material, and is a function of permeability and the absorptive power of the material. This ability is interesting for example when studying the robustness of a

Table 3.3 Properties of some materials

Materials	Porosity	Specific area (m^{-1})	Permeability (m^2)
Brick	0.12–0.34		4.8×10^{-15} to 2.2×10^{-13}
Granular catalyst	0.45	5.6×10^{7}	
Standard concrete	0.02–0.07		
Fiberglass	0.88–0.93	56×10^{3} to 77×10^{3}	
Sand	0.37–0.50	15×10^{3} to 22×10^{3}	2×10^{-11} to 1.8×10^{-10}
Silica (powder)	0.37–0.49	6.8×10^{5} to 8.9×10^{5}	1.3×10^{-14} to 5.1×10^{-14}

construction against interstitial condensation. Duforestel (1992) proposed an empirical model to calculate the indoor moisture buffer, and classified the hygroscopic materials into two general categories: high absorption and low absorption. The Nordetest protocol proposes to place square samples to the wetting and drying cycles clearly defined (Rode et al., 2007). The moisture-buffering capacity can be determined experimentally by weighing the samples, which are subjected to periodic changes in humidity:

$$\mathrm{MBV}_{8\mathrm{h}} = \frac{m_{\max} - m_{\min}}{A \cdot \left(\mathrm{RH}_{\text{humidification}} - \mathrm{RH}_{\text{drying}}\right)} \tag{3.13}$$

where $m_{\max}$ and $m_{\min}$ are the maximum and the minimum water mass accumulated in the sample, respectively, A (m^2) is the exposed specimen surface, $\mathrm{RH}_{\text{humidification}}(\%)$ is the relative humidity applied during the humidification cycle, and $\mathrm{RH}_{\text{drying}}(\%)$ is the applied relative humidity during the drying cycle.

Moisture buffer values (MBVs) using the Eq. (3.13) and the results of successive cycles of wetting and drying (regular weighing was performed for each cycle) are shown in Fig. 3.10 for aerated concrete, oriented strand board (OSB), and wood fibrous insulation (WFI).

According to the MBV classification given by Rode (2005), the buffering effect of building materials can be considered as very important for the durability and for hygrothermal performance of a building. As for thermal diffusivity, the moisture accumulation capacity can be expressed as follows (Hagentoft, 2002):

$$b_{\mathrm{m}} = \sqrt{\frac{\delta_{\mathrm{p}} \cdot \rho_{\mathrm{s}} \cdot \xi}{p_{\mathrm{sat}}}} \tag{3.14}$$

b_{m} ($\mathrm{kg/m^2\,Pa\,s^{0.5}}$) is the moisture accumulation capacity, ξ is the slope of the sorption desorption isotherm, ρ_{s} ($\mathrm{kg/m^3}$) is the dry density of the sample, δ_{p} ($\mathrm{kg/(m\,s\,Pa)}$) is the vapor permeability, and p_{sat} (Pa) is the saturation pressure.

Such a secondary evaluation method is performed by using the moisture accumulation calculated by Eq. (3.14). It is mainly based on the experimental results of sorption desorption isotherms of the studied materials. The results are shown in Fig. 3.11. The

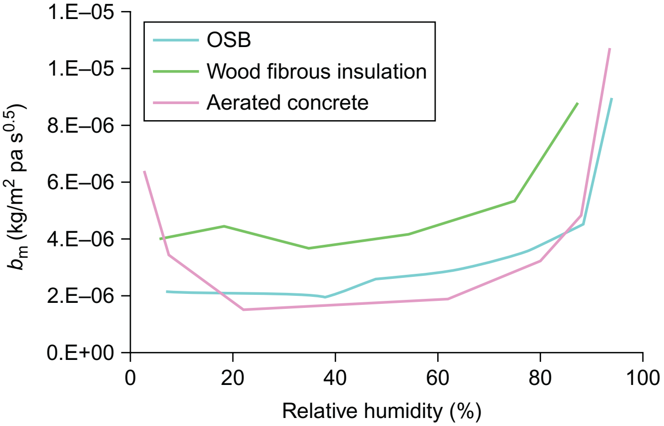

Fig. 3.11 Moisture accumulation capacity b_{m} as a function of relative humidity ($\mathrm{kg/m^2\,Pa\,s^{0.5}}$).

ability of the materials to exchange moisture with the ambient air (ie, buffer capacity) has been explained using two different methods (Eqs. 3.13, 3.14) (Peuhkuri, 2003). The most significant observation is that for both methods, the same classification of the behavior of materials is obtained.

Both buffering capacity assessment methods are adapted to conditions at equilibrium, unlike the dynamic method that has been proposed by Peuhkuri (2003). It was shown that, in general, the moisture-buffering capacity is sometimes underestimated when using equilibrium conditions with respect to the dynamic approach.

3.7 Durability of bio-based material

In the previous sections, the focus was mainly on the study of the cementitious material's durability as a function of the morphological and the local physical properties. However, nowadays, bio-based materials are becoming increasingly competitive to fully cementitious materials due to their environmental benefits.

Actually, wooden or partially wooden materials present many advantages; in addition to being a 100% renewable resource, wood is a naturally recyclable and biodegradable building material. Compared to other building products like concrete and steel, it has a lower environmental impact (Bribián et al., 2011). Because of their complex morphology and the high heterogeneity of wooden materials, there is still a lack of knowledge concerning their microscopic geometric changes, due to the hygrothermal solicitations. These changes usually lead to a considerable evolution of the resulting thermophysical properties of the material, which constrain its use. Indeed, the latewood/earlywood transition present in the wood structure substantially influences the macroscopic behavior of the material.

In reality, the hygrothermal behavior of wood involves the complex interaction of different heat, air, and moisture transfer mechanisms. At the pore (1–100 μm) scale, the physical transfer as well as the liquid–vapor interface topology and their interactions with the solid matrix make the study of these mechanisms extremely complex. In fact, the macroscopic material's behavior is highly dependent on the mechanisms involved at the microscopic scale. Indeed, the swelling and shrinkage phenomena, observed at the macroscopic scale, is the main evidence reflecting such change caused by the hygric transfer history within the material; unfortunately these dimensionless phenomena are often neglected by the researchers.

Recently, El Hachem et al. (2015) conducted an experimental investigation on spruce wood based on the nondestructive, 3D-imaging technique "X-ray tomography." This method allows for the locating of any heterogeneity in the material (see Fig. 3.12, obtained at a resolution of $3.35 \times 3.35 \times 3.35\,\mu\text{m}^3$ at dried conditions).

The region of interest (ROI) of a dried specimen containing the latewood/earlywood discontinuity is represented in Fig. 3.5 (above section). In both figures, it can be observed that the latewood phase is characterized by the cells containing thicker walls and smaller cavities that become smaller when approaching the latewood/earlywood interface. It can be noticed that the smallest pores belong to the thickest fibers.

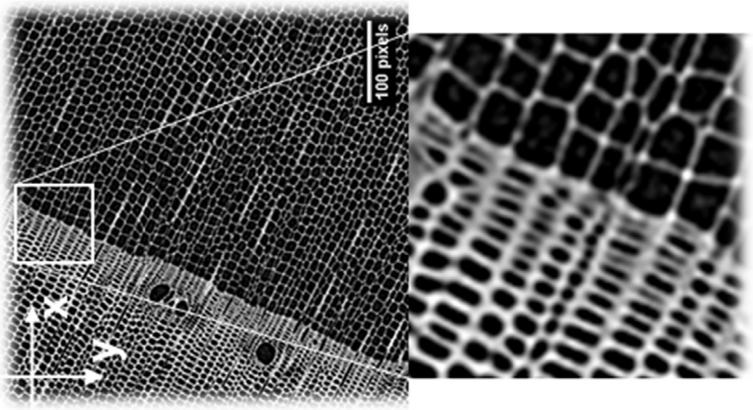

Fig. 3.12 Region of interest of a dried spruce wood specimen in the (*x*,*y*) plane, without the latewood/earlywood interface presence.

El Hachem et al. (2015) confirmed that the presence of the latewood/earlywood discontinuity in the material decreases its porosity from 80% to 50%. In addition to these wooden material's morphological changes and their interaction with mortar on the interface inclusion, these materials are exposed to other risks affecting their durability. Such risks must include several criteria, which make them hazardous; mold growth is one of the most important criteria for the evaluation of the material's degradation since the porous parts of these bio-based materials are continuously exposed to internal and external environmental conditions (see Fig. 3.13).

Currently there are several known adverse health effects due to the exposure to fungi. For example, immunological effects that are associated with allergic reactions, subsequent to the exposure of allergenic proteins; some types of fungi such as, Cladosporium, Alternaria, Penicillium, Aspergillus, are commonly associated with allergies and aggravate rhinitis and asthma (Boudih, 2011). Thus, there is a necessity to examine such fungal phenomenon that may be continuously present in our buildings. Moisture is often the most influential parameter for spore germination and mold growth. Water availability is required for fungi to ensure their physiological and metabolic activities. However, fungal growth depends not only on the ambient relative humidity, but also on the water activity of the substrate.

In this context, the mold-growth behaviors of three wooden material types (massive spruce wood (MSW), WFI, and OSB) are examined. Fig. 3.14 shows an example of mold growth for the MSW, WFI, and OSB samples exposed at a 99.4% of relative humidity in an isothermal desiccator (at 25°C).

After just 1 week of exposure, the contamination surface was visually detected on treated (sterilized) and untreated OSB samples. This is certainly linked to the chemical composition of OSB (its wealth of different types of wood) and its high heterogeneity. It also confirms the results of Lähdesmäki et al. (2011), which assume that fungi prefer to grow on a rougher surface than smooth; the OSB surface is rougher than that

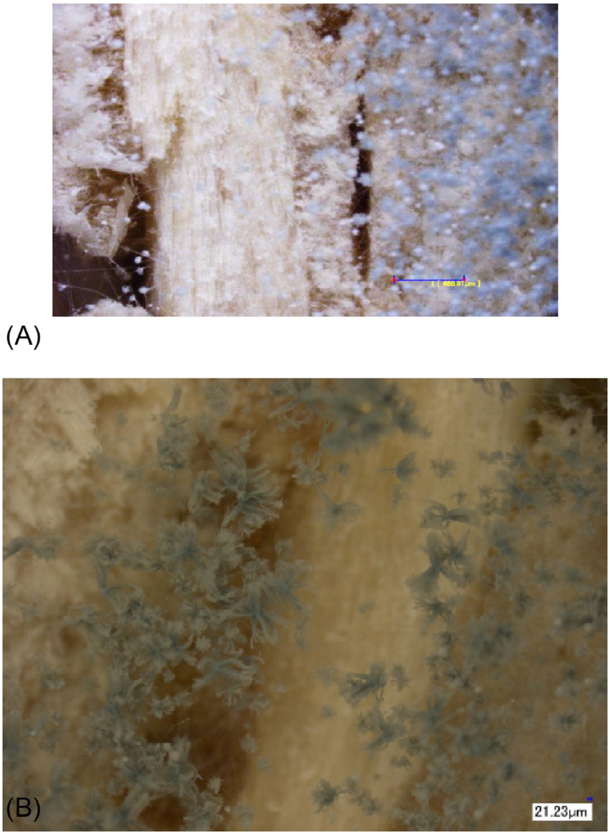

Fig. 3.13 Mold observation for oriented strand board (A) and wood fibrous insulation (B) after 2 weeks of exposure to 80% of relative humidity. Observation under the Keyence microscope.

of WFI and has a rougher surface than that of MSW. The results also confirm that the untreated material is more susceptible to develop fungal compared to those sterilized initially.

High sensitivity to the hydric changes was obtained in particular for OSB, which presents the most heterogeneous composition compared to MSW and WFI; it presents a maximum mold index value (noted M) of 6 observed visually at high relative humidity (99%) after only 1 week of exposition, which presents a high risk of early degradation (Fig. 3.14). We indicate here that the index-classification criteria (M) have been recently modified to allow its application to the evaluation a large type of materials (Lähdesmäki et al., 2011). For example, mold indexes of 2 signify moderate growth detected with microscopy, and a mold index of 5 signifies a visually detected growth of more than 5%. This parameter can be used as input data for material models (Hukka and Viitanen, 1999) predicting the response of pure-wooden material to arbitrary temperature and humidity conditions.

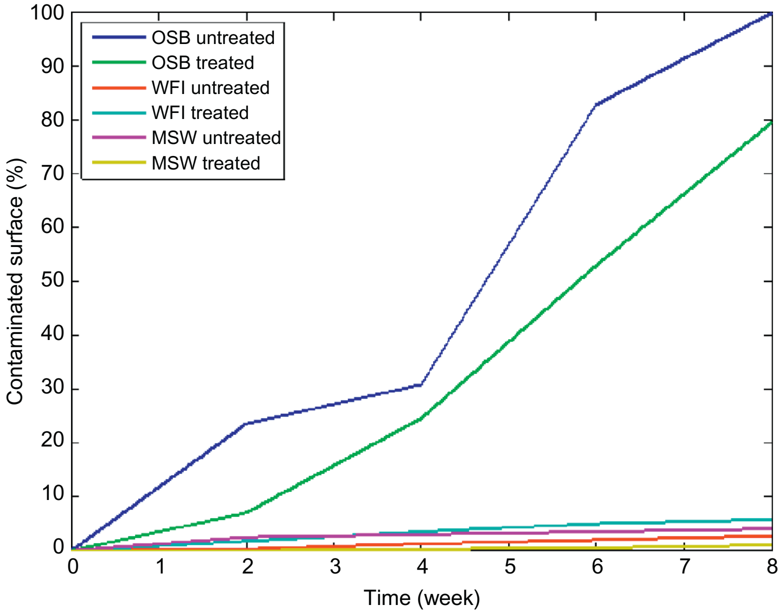

Fig. 3.14 The mold of OSB, WFI, and MSW samples conditioned in transparent desiccator at 99.4%.

3.8 Future trends

This chapter focuses on the effect of physical parameters such as porosity and permeability on the durability and thermal properties of construction materials as well as the effect on the environment and human health.

The amplification of permeability by cracks was also considered. The water-vapor transfer and adsorption phenomena constitute an important characteristic for some materials, and exhibit their great ability in water storage. Such water-storage abilities affect the diffusion phenomena, due to multiphase coupling, and also allow for the interaction of the material with the ambient vicinity. Such interactions provide the environmental-regulation properties (ie, the moisture-buffering capacity) of materials. These parameters characterize the ability of a material to moderate the variations in relative humidity of the surrounding air.

The inclusion of biomaterials increase the complexity of the interaction on a multiscale and modify the durability task. Such interaction is due to the different time-scale difference and the strong inhomogeneity between the biomaterials and the cement paste. Such inhomogeneity affects the global durability and the water or humidity distribution in such innovative construction materials. Such new problems contribute significantly to fungi development and allow for improved knowledge of fungi's effect on people's health.

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Nanotechnologies for sustainable construction

4

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4.1 Introduction

Nanotechnology has been gaining popularity among the industrial sector and researchers in recent decades. The number of products containing nanomaterials that enter the market has also increased rapidly, and this trend is going to be even more pronounced in the coming years. The total value of nanotechnology products in the market is estimated to grow to $3000 billion in 2020 (Roco, 2011). The reason for this favorable trend is the unique and interesting properties that nanoparticles exhibit.

Nanotechnology has resulted in a variety of products and applications. Along with various benefits in the industrial sector, the construction sector has observed significant progress in the past few decades in the field of nanotechnology. Because of the dimensions controlled in the transitional zone between atom and molecule, nanomaterials can contribute to traditional building materials with extra functionalities, including, for example, enhanced durability and mechanical properties (Lee et al., 2009). The novel properties introduced by nanomaterials could provide additional performance, for instance, adjustable conductivity or air-purifying property for traditional building materials like concrete, the most common man-made material in the world.

4.1.1 Nanotechnology

According to the National Nanotechnology Initiative in the United States: Nanotechnology is science, engineering, and technology conducted at the nanoscale, which is about 1–100 nm. The lower and upper limits of this range are selected by convention. This range is above the size of atoms (0.2 nm is the size of an H atom) and below the size that phenomena related to nanomaterials start to be evident. To set those sizes in context, the size ratio between a nanometer and a meter is the same as that of a marble to the Earth. Another interesting definition was given by Roco et al. (1999), who expressed: "Nanotechnology is the ability to control and restructure the matter at the atomic and molecular levels in the range of approximately 1–100 nm, and exploiting the distinct properties and phenomena at that scale as compared to those associated with single atoms or molecules or bulk behavior. The aim is to create materials, devices, and systems with fundamentally new properties and functions by engineering their small structure. This is the ultimate frontier to economically change materials properties, and the most efficient length scale for manufacturing and molecular medicine."

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Nanoparticles have always been present in the environment. Additionally, humans have also been working with nanoparticles (eg, natural waxes, clays, paints) since ancient times, possibly even before. The big difference is that now we have a better understanding of the physical and chemical phenomena of nanotechnologies. As a consequence of this new knowledge, materials with new properties can be formulated and produced, allowing researchers to take advantage of the special properties that occur in the nanoscale.

Ideas and concepts about nanotechnology were first discussed in 1959 by Richard Feynman in his talk "There's Plenty of Room at the Bottom," in which he described the possibility of material synthesis via direct manipulation of atoms. However, it was not until 1974 that the term *nanotechnology* was first used by Norio Taniguchi (Taniguchi, 1974). Stimulated by Feynman's ideas, Drexler used the term *nanotechnology* in his book *Engines of Creation: The Coming Era of Nanotechnology* (Drexler and Minsky, 1990), proposing the idea of a nanoscale self-assembler particle, which could build a copy of itself and of other items with atomic control.

4.1.2 Advances in nanotechnology in recent years

Table 4.1 reports five key indicators about the development of nanotechnology in the world (Roco, 2011). These indicators are (1) the number of people working in this sector; (2) the number of scientific publications in journals; (3) the number of filed patents; (4) the amount of final products consumed in the market; and (5) the funding for research and development. The value of 2015 in Table 4.1 is an estimate, and the value of 2020 is an extrapolation. The fast growth of this sector is remarkable despite the global crisis that occurred in 2008 and 2009. According to Roco's study, the values of the workforce and the final products in the market will have increased by 100 times in only 20 years. In terms of research, between 2000 and 2008, the number of articles has quadrupled, while patents and the R&D funding increased tenfold.

4.1.3 Special properties of nanotechnology

While size plays an important role in the properties of nanomaterials, materials in the micro- and macrorange (also called bulk materials) have constant physical properties

Table 4.1 Indicators of nanotechnology development in the world (Roco, 2011)

Year	Primary workforce	SCI papers	Patents	Final products market ($billions)	R&D funding ($billions)
2000	60K	18,085	1197	30	1.2
2008	400K	70,287	12,776	200	15
2015	2000K	–	–	1000	–
2020	6000K	–	–	3000	–

The values of 2015 and 2020 are an estimate and extrapolation, respectively.

regardless of their size. Several phenomena affecting the properties of materials appear as the size of particles is reduced below the microrange. These phenomena are due to quantum mechanical effects and to other physical effects (eg, high specific surface area). In addition, because the majority of biological processes occur at the nanoscale (eg, a DNA molecule has a diameter of 2 nm), nanotechnology has been also used in this discipline, resulting in many applications in the fields of biology and medicine.

Quantum effects dominate the behavior and properties of materials at sizes below 100 nm. At this dimension, the particle size has a great influence on the material properties; for example, size can strongly impact melting point, fluorescence, chemical reactivity, electrical conductivity, etc. Some examples of these different behaviors are (1) the red (or purple) color of gold due to the confinement of electrons in much smaller sizes, causing the gold nanoparticles to interact with light in a different way than the bulk material (Daniel and Astruc, 2004); and (2) the lowering of a melting point with the reduction of the particle size (eg, 2.5-nm gold nanoparticles melt at around 300°C instead of at 1064°C) (Buffat and Borel, 1976).

Specific surface area increases exponentially with the size reduction of particles. The amount of atoms at the surface compared to the number of atoms in the inner part becomes significant for nanomaterials, increasing the amount of material exposed to the surrounding medium, and therefore making nanomaterials much more reactive than bulk materials. Some examples of these difference properties due to the increase of specific surface area are (1) the solubility change of silica with the particle size; the smaller the particle, the higher the solubility (Iler, 1979); (2) insoluble particles become soluble such as with gold particles (Daniel and Astruc, 2004); and (3) materials like aluminum that are stable at normal conditions but can turn combustible when the particle size is reduced.

The properties of nanomaterials do not always have positive effects, being sometimes even harmful to human health, animal health, and/or the environment. Some known examples of negative effects are (1) nanofibers, which are used in different commercial products, leading to pulmonary illnesses (Byrne and Baugh, 2008); (2) TiO_2 nanoparticles penetrating the skin of hairless mice causing different pathological lesions in several organs (Wu et al., 2009); and (3) SiO_2 nanoparticles increasing carcinogenic risk (International Agency for Research on Cancer, 1997) in humans and animals as well as being hazardous to bacteria (Adams et al., 2006). The main point here is that the same properties that make nanomaterials so promising may also have negative effects for human health and the environment. Therefore, knowledge of the nanomaterials' properties and the interaction between nanomaterials and the environment is fundamental for taking advantage of the benefits of these materials while at the same time avoiding their harmful effects.

4.1.4 Sustainable construction

Construction materials, a term referring to materials used in construction and building, can be classified based on their type into categories such as metal (eg, steel, aluminum), mineral (eg, natural stone, concrete, gypsum, glass), or organic (eg, plastic, bitumen). The construction sector is critical to the world economic system, and it contributed

Table 4.2 **Annual global production of materials in 2008 (Brouwers, 2010)**

Material	Amount (ton)	Material	Amount (ton)
Timber	4000×10^6	Quicklime	130×10^6
Plastics and rubber	250×10^6	Glass	120×10^6
Steel	1400×10^6	Cement	2500×10^6
Gypsum	250×10^6	Concrete	$15{,}000 \times 10^6$

to US $3.3 trillion to the global economy in 2008 (Provis and van Deventer, 2014). Table 4.2 lists the global production of the most used man-made construction materials in 2008. These enormous amounts indicate the great consumption of energy for the production and transport of raw materials and products.

Among all the previously listed materials, concrete is by far the most used man-made building material because it is cheap, easy to cast to any shape according to requirements, mechanically strong and durable. Concrete normally has five main constituents: cement, aggregates, water, additives, and chemical admixtures, among which cement plays the crucial role of the binding agent. Portland cement (PC) is the world's most used binder for production of construction materials. Due to its good mechanical properties, relatively low cost, good durability, and availability of the raw materials, PC concrete is favored in many applications. Nevertheless, the production of PC has some major drawbacks; for example, depletion of natural habitat and fossil fuels, and high emissions of CO_2 and other greenhouse gases (Provis and van Deventer, 2014). Such an enormous volume of production is associated with a very significant environmental problem because cement production contributes at least 5–8% of global CO_2 emission. Nevertheless, it is very difficult to reduce this negative impact since about 90% of the energy needed for concrete production is spent in the production of cement (Hüsken and Brouwers, 2008).

Therefore, increasing interest in searching for alternative systems such as replacing cement with other supplementary cementitious materials and developing new binders such as alkali-activated materials (AAMs) are observed (Provis and van Deventer, 2014). These materials may be beneficial in terms of overall sustainability characteristics by applying industrial by-products as a partial precursor material instead of a primary raw mineral binder like PC. Additionally, depending on the applied raw materials and alkali activators, the end-products could show better properties compared to PC concrete, such as lower hydration heat, lower thermal conductivity (TC), and enhanced durability (Davidovits, 1989, 2002; Rangan, 2010; Wallah and Rangan, 2006).

Furthermore, applying recycled or waste materials as aggregates in concrete brings further effects besides environmental benefits, for instance, sustainability by reducing the utilization of natural sources. There already exist various successful examples of such development, including recycled concrete aggregates, granulated blast furnace slags, municipal solid waste incineration bottom ashes, fly ashes, etc.

4.2 Nanotechnology for sustainable construction

Nanotechnology has extensively been applied in construction materials with novel functionalities and improved characteristics. The application of nanomaterials can be found in many materials, including cement, concrete, coating, paint, glass, etc. In concrete, nano/microsilica has been widely investigated and applied (García-Taengua et al., 2015; Lazaro et al., 2013a; Quercia Bianchi, 2014; Quercia et al., 2013; Sonebi et al., 2015). Improved performances such as mechanical properties, durability and elasticity are reported, and reduction of energy consumption has been claimed due to the reduced cement dosage and due to the improved insulation when applied as façade materials.

In recent years, great attention has been paid to air quality, including both indoors and outdoors. Application of nanotechnology for this purpose, for example, photocatalytic oxidation (PCO) technique, has been addressed by numerous studies. This is reflected in the many published articles in the last decade. A semiconductor (eg, TiO_2) can be applied in the matrix of construction materials like concrete or on the surface of the construction materials in a format of coating. Air pollutants, including both organic (eg, volatile organic compounds (VOCs) like toluene) and inorganic (eg, NO_x and SO_x), can be removed by photocatalysis. Furthermore, microorganisms and particles can also be degraded by PCO. In addition, the photo-induced superhydrophilic property of TiO_2 brings an extra dimension of this technology, namely, a self-cleaning property. A successful example is a TiO_2 surface-modified glass window, which possesses an antifogging property because of the light-mediated surface hydroxylation.

Another example of nanomaterials in construction materials is carbon-based nanomaterials. Carbon nanotubes as a proxy for polymeric chemical admixtures can remarkably improve certain properties such as durability by gluing concrete ingredients. Nanosensors are also implemented in construction structure for various purposes, among which one important application is health monitoring (can be remotely controlled and real-time monitored). Another example is the synthesis and application of nanosilica in construction materials. Nanosilica particles can modify light absorption/reflection, fire resistance, durability, and mechanical properties of the implanted materials.

It can be primarily concluded that many improvements have been achieved due to application of nanotechnology. Nevertheless, it can also be observed that compared to the advancement of nanotechnology and nanomaterials in other fields, the construction sector has been relatively slow in catching up with the revolution of nanotechnology. Reasons for this may include (Garboczi, 2009): (1) the lack of fundamental understanding of the chemical and physical mechanisms and structure of nanomaterials at the nanorange, without which any attempted modifications at this length scale will only be empirically based; (2) lack of insight on what modifications nanomaterials will bring to construction materials, especially very complex materials like concrete. Hence, a promising but still long way is in front of us to dig into the advancement of nanotechnology in construction materials.

4.3 Health and environmental risks

Nanoparticles have always been present in the environment, and therefore humans have been exposed to them. Some examples are blood (solid nanoparticles in liquid), milk (fat nanoparticles in water), and volcanic ash (solid particles in air). The first two examples illustrate beneficial nanoparticles for human beings and animals. Ashes together with other solids dispersed in air (ie, aerosols) can be problematic to human health; however, in most cases of human exposure to naturally present aerosols, that is not the case since human physiology is capable of protecting itself against unwanted substances. In fact, viruses and nonbiological particles are usually eliminated from the body by the reticuloendothelial system (Buzea et al., 2007).

Anthropogenic nanoparticles have also been produced for several centuries, but the amount of released nanoparticles has increased tremendously in recent decades mainly due to more industrial development and the mass use of combustion-based engines. An example of the consequences of the massive use of vehicles was demonstrated by Shi et al. (2001). Shi showed that the percentage of particles below 10 nm encountered at the roadside was as high as 40%. Technological advancement has not only increased the concentration of nanoparticles but also enlarged the diversity of nanoparticles present in the environment. Nowadays, there are already many products that contain nanomaterials in their recipes, such as sunscreen with nano TiO_2, green tires with nano SiO_2 and concrete with silica fume (containing micro- and nanosilica particles). Although there are already products in the market containing nanoparticles, there is no clear and harmonized legislation in Europe or the United States about the use of nanoparticles.

The exposure of living organisms to nanoparticles can be harmful to their health. Two of the main reasons for the danger of nanoparticles are (1) that the small size of these particles allows them to penetrate the structure of some cells, disrupting their normal function; and (2) nanoparticles can be translocated to several tissues and organs, including the brain (Buzea et al., 2007; Oberdörster et al., 2004). After exposure to nanoparticles, some of the following diseases or malfunctions can be observed: (1) oxidative stress (Churg, 2003; Jonaitis et al., 2010; Wu et al., 2009), (2) inflammation of tissues and pulmonary diseases (Byrne and Baugh, 2008; Oberdörster, 2000; Oberdörster et al., 2002; Sato and Donaldson, 1998; Sayes et al., 2006), (3) cytotoxicity (Reeves et al., 2008; Sayes et al., 2006), (4) DNA damage (Donaldson et al., 2005; Peters et al., 2006; Reeves et al., 2008; Risom et al., 2005), (5) carcinogenic effects (Borm et al., 2004; International Agency for Research on Cancer, 1997; Knaapen et al., 2004; Lin et al., 2006), (6) neurological diseases (Brown et al., 2005), and (7) heart diseases (Delfino et al., 2005).

Table 4.3 presents the main toxicological impacts of nanotitania, nanosilica, and copper nanoparticles, which are common nanomaterials used in the construction industry. This table has been adapted and updated from the review article of Lee et al. (2010).

Table 4.3 Toxicity of titania, silica, and copper nanoparticles

Nanomaterial	Toxicological effects	Reference
TiO_2	Acute lethality Growth inhibition Bactericidal for Gram-positive bacteria Suppression of photosynthetic activity Oxidative damage due to ROS (reactive oxygen species)	Adams et al. (2006), Aruoja et al. (2009), Blaise et al. (2008), Jin et al. (2011), Kasemets et al. (2009), Rincon and Pulgarin (2004), Wang et al. (2008), and Wolfrum et al. (2002)
SiO_2	Mild toxicity due to TOS (toxic oil syndrome) production Toxic to marine algae Apoptosis Upregulation of tumor necrosis factor Inflammatory and immune responses	Adams et al. (2006), Dutta et al. (2007), Fujiwara et al. (2008), Lucarelli et al. (2004), Sergent et al. (2012), Yu et al. (2012), and Yu et al. (2011)
Cu nanoparticles	Toxic to freshwater algae Toxic to yeast DNA damage (single-strand breaks) Lipid peroxidation Acute toxicity to liver, kidney, and spleen	Aruoja et al. (2009), Blaise et al. (2008), Bondarenko et al. (2013), Chen et al. (2006), Kasemets et al. (2009), and Midander et al. (2009)

4.4 Selected examples of green nanoconstruction

4.4.1 Synthesis of nanosilica via a sustainable route

An alternative synthesis route to the commercial production methods of amorphous nanosilica is the dissolution of silicate minerals in acid (Lazaro et al., 2012; Lieftink, 1997). The raw materials employed in this method are olivine ($(Mg,Fe)_2SiO_4$) and acid (in the present case, sulfuric acid). The dissolution of olivine in acid at low temperatures (between 50°C and 95°C) produces amorphous silica in a relatively short time (more information about the kinetics of the dissolution of olivine can be found in Lazaro et al., 2015):

$$(\mathrm{Mg,\ Fe})_2\,\mathrm{SiO_4} + 4\mathrm{H^+} \rightarrow \mathrm{Si(OH)_4} + 2(\mathrm{Mg,\ Fe})^{2+}$$

The olivine nanosilica (ONS) prepared has a specific surface area between 100 and 500 m^2/g. The silica particles are agglomerated in clusters forming a material with micropores (pores below 2 nm), mesopores (pores between 2 and 50 nm), and macropores (pores above 50 nm). The purity content is above 99% and can be further cleaned by performing additional washing steps (more details can be found in Lazaro et al., 2012; Lazaro et al., 2013b; Lazaro, 2014). A TEM (transmission electron microscope) photograph of this type of silica is presented in Fig. 4.1.

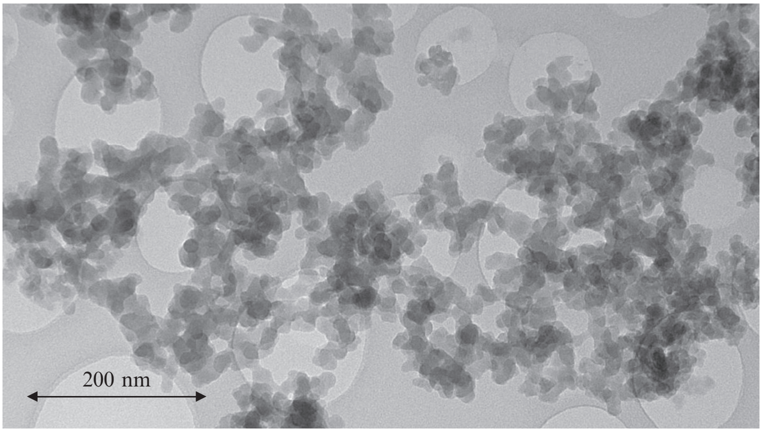

Fig. 4.1 TEM picture (89 kx) of the ONS (Lazaro et al., 2012).

Table 4.4 Energy generation during the ONS process

ΔH_r (kJ/mol)	$V_{H_2SO_4}$ (L)	$m_{H_2SO_4}$ (g)	n_{ol} (mol)	m_{du} (g)	X (%)	Q (kJ)	ΔT (°C)
223	1	1186	1.5	242	100	333.5	79.3

ΔH_r is the enthalpy of reaction, X the conversion degree of the reaction, and Q is the heat generated. Heat capacities of 25% sulfuric acid and forsterite are 3.38 (Perry and Green, 2008) and 0.83 (Robie et al., 1982) J/(g °C), respectively. The heat capacity of dunite has been approximated as the one of forsterite.

In addition to the low temperature of this procedure (below 95°C), this process is exothermic with a reaction heat of 223 kJ/mol of olivine (Jonckbloedt, 1997). The energy generation during the ONS process for the hypothetical case of an adiabatic reactor is shown in Table 4.4. When 1.5 mol of olivine react with a 1-L solution of sulfuric acid 25%, the temperature of the mixture will increase to 79°C. Therefore, the reaction generates more than enough energy to keep the system at the desired temperature (between 50°C and 90°C) provided the reactor is sufficiently insulated. In addition, the dissolution of concentrated sulfuric acid also generates a considerable amount of heat (Lazaro et al., 2013a).

The energy required during the synthesis of ONS could be considered negligible since the reaction is exothermic, provided the reactor is sufficiently large and well insulated. On the contrary, the energy consumption and the CO_2 emission of precipitated nanosilica produced using sodium silica solutions are huge due to the high temperatures used in the production of sodium silicate solution and the release of CO_2 from Na_2CO_3. The actual values depend on several factors such as the energy source, efficiency of the installation, raw materials, etc. An estimation of energy requirements and CO_2 emissions of precipitated nanosilica can be obtained from the values of sodium silicate solutions (European Commission, 2007), resulting in 2600 MJ/ton and 1690 CO_2/ton. Thus, the energy requirements and CO_2

emissions are much lower for the ONS process than for precipitated nanosilica, making it one of the most promising materials for reducing the CO_2 footprint in the concrete field.

4.4.2 Replacement of cement by other materials such as pozzolanic materials, nanosilica, nanofillers

Nanosilica in concrete is not yet commonly applied, but silica fume, which is considered a microsilica, has already been used in concrete for several decades to produce high-performance concrete. Due to its considerably fine particle size, micro/nanosilica has been observed to accelerate the hydration process at early ages (Shaikh et al., 2014), refine the pore structure (Said et al., 2012; Zhang and Li, 2011), and enhance the mechanical properties even at small levels of replacement (Shih et al., 2006). Moreover, micro/nanosilica also exhibits ideal pozzolanic activity owing to its amorphous nature and high specific surface area (Björnström et al., 2004; Qing et al., 2007), which lead to the formation of additional C-S-H gel by reacting with calcium hydroxide and result in a denser matrix (Gaitero et al., 2008; Lin et al., 2008; Schmidt et al., 2013). The use of microsilica in concrete continues to increase despite its relatively high cost because of its pozzolanic behavior and its content of fine particles. These two features of the microsilica confer some benefits to the concrete. Also, because of its small particle size, microsilica fills the voids between the cement particles; this improves the packing and reduces the porosity. Besides these features, nanosilica has the following effects on cement pastes and concrete mixes: acceleration of the setting, cement matrix densification and improvement of the interparticle transition zone (ITZ) of aggregates (filling effect).

One of the reasons to add microsilica to concrete is to replace the cement content with microsilica. Microsilica can replace cement due to its pozzolanic reaction (1 part silica instead of 3–4 parts cement), while the strength is unaffected (Vijayarethinam, 2009). Therefore, the replacement of cement by microsilica should considerably reduce the CO_2 footprint of concrete. That is important because the cement industry is one of the industrial sectors that releases large amounts of CO_2 into the environment, accounting for 8% of global CO_2 emissions (PBL Netherlands Environmental Assessment Agency, 2012). In addition to this interesting application, the largest use of microsilica is for producing concrete with enhanced properties, such as high early strength, low permeability and enhanced durability (Spiesz, 2013). Considering that the main difference between nanosilica and microsilica is their respective particle sizes and assuming that the pozzolanic behaviors in both materials are similar, nanosilica will react faster with the cement than microsilica due to its higher SSA (specific surface area).

The effect of ONS in conventional vibrated concrete (CVC), which is the most commonly used concrete, was investigated (Lazaro et al., 2013a) by casting three mixtures with different replacement levels of CEM I 52.5 N with ONS (5%, 7%, and 10% by volume). The ONS was applied in concrete in a water dispersion form. This dispersion was prepared using the high shear mixer (Silverson multifunctional L5M laboratory mixer with a shear screen) for about 10 min. The mix designs were based

Table 4.5 Mix designs of CVC with and without replacement of cement with ONS

Materials (kg/m³)	Reference	5% vol.	7% vol.	10% vol.
Olivine nanosilica	0	6.9	10.3	13.7
CEM I 52.5 N	210	199.6	194.3	189.1
Fly ash	88.2	88.2	88.2	88.2
Sand 0-4	780.6	780.6	780.6	780.6
Gravel 4-16	1086.3	1086.3	1086.3	1086.3
Water	158.8	159.1	158.1	158.1
SP (% BWOB, based on the weight of binder)	0.5	1.12	1.33	1.75
w/f (%)	0.54	0.54	0.54	0.54
Slump class (mm)	S2 (60)	S2 (60)	S1 (40)	S1 (40)

Where w/f refers to the ratio of water to fines.

on a commercial recipe (see Table 4.5); 18 cubes were cast using a vibrating table and were tested for their compressive strength after 1, 7, and 28 days. Additionally, the workability of the fresh concrete was investigated by analyzing the slump following 206-1 standard (DIN, 2000). The superplasticizer (SP) used was Ha-BE 100 (polycarboxylate ether type). Table 4.5 also presents the results of the slump test. The only mix with similar slump values to the reference mix was the one with 5% replacement by volume. The SP requirement for this mix was more than double compared to the reference mix. In the cases of 7% and 10% replacement, even though the SP contents were higher than for concrete with 5% replacement, it was not possible to obtain the desired slump class. Therefore, when the specific surface area of the mix was increased by addition of nanosilica, more SP was required to maintain the same slump class. This is a clear disadvantage of the use of nanosilica, and it needs to be addressed in the future in order to find the type of SP that works more efficiently with ONS. Another possible solution for this problem could be to tailor the properties of ONS to get lower specific surface areas and more spherical particles.

The compressive strengths of concrete after 1, 7, and 28 days are depicted in Fig. 4.2. This figure shows that the strength after one day was not significantly affected by the increase of the SP content in these mixes. Only the mix with 10% replacement showed a lower strength than the reference. The 7-day compressive strength, on the other hand, displayed an increase for all the substitution levels. The 28-day compressive strength showed similar trends as the 1-day compressive strength; only the mix with 10% replacement showed a lower strength than the reference. The best result after 28 days was obtained for the mix with 5% replacement, where the compressive strength increased by 20% compared to the reference mix. This suggests that the optimum substitution of ONS should be around this value.

Fig. 4.3 presents the estimated CO_2 footprint per cubic meter of reference CVC and CVC with 5% replacement. These estimations were performed using the CO_2 footprint of each compound from a database of the Dutch precast concrete organization. The

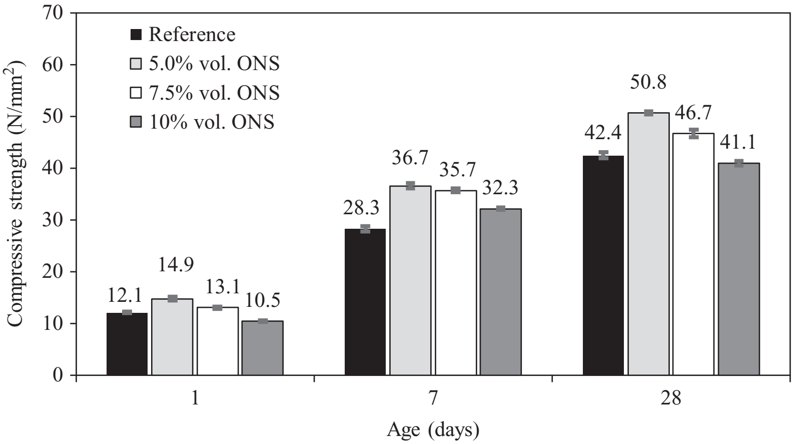

Fig. 4.2 Compressive strength development of CVC at different replacement levels of cement with ONS.

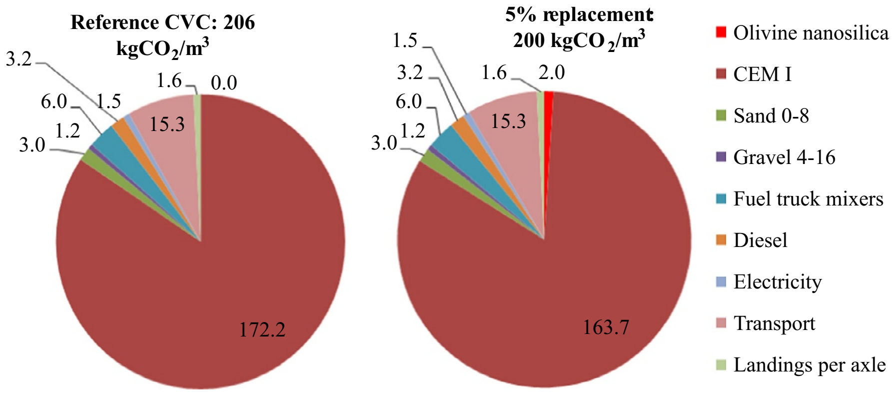

Fig. 4.3 CO_2 footprint of CVC and of CVC with replacement of cement with ONS.

CO_2 footprint of olivine was estimated from a life-cycle analysis performed by Paju et al. (2012). The reduction of CO_2 emissions for CVC with 5% replacement was 3% with respect to the reference concrete. This could be further improved by tailoring the properties of ONS so less SP would be necessary to maintain the same rheological properties or slump class. Since the compressive strength of CVC with 5% replacement was 20% higher than the reference concrete, there would also be the possibility of reducing the total amount of cement used while maintaining the same compressive strength as the reference material, therefore minimizing CO_2 emissions.

4.4.3 Nanotechnology in AAMs

In order to reduce the negative environmental impacts of cement industry, great attention has been paid to apply AAMs as alternatives to PC. This type of material generally exhibits superior mechanical properties (Wang et al., 1994), durability

(Bakharev et al., 2003; Fernández-Jiménez et al., 2007), and thermal resistance (Zhang et al., 2014) together with significantly reduced energy costs and carbon emissions (Rashad, 2013a; Shi et al., 2011). Based on the chemical composition of the raw materials, two types of binding systems can be classified: One is the alkali-activated slag (Si + Ca) system, having a C-A-S-H type gel with a low Ca/Si ratio and a high Al incorporation as the main reaction product (Brough and Atkinson, 2002); the other is the alkali-activated class F fly ash/metakaolin (Si + Al) system, having N-A-S-H type gels with three-dimensional network as the major reaction products (Li et al., 2010). Both systems exhibit distinct behaviors regarding alkaline and curing demands, setting and hardening processes and strength development due to their differences in reaction mechanism and gel characteristics.

Recently, growing interest has focused on the blended alkaline system (Na_2O–CaO–Al_2O_3–SiO_2 system) that is prepared by mixing calcium-enriched precursors and low-calcium aluminosilicates. Compared to the individual system, the blended system shows a better control of setting times (Lee and Lee, 2013), as well as modified workability (Rashad, 2013b), shrinkage (Aydin, 2013), mechanical properties, and durability (Sugama et al., 2005). Microscale analysis reveals that the reaction products are stably coexisting C-(A)-S-H and N-A-S-H type gels with higher degree of cross-linking (Ismail et al., 2014; Lodeiro et al., 2010; Yip et al., 2005), indicating a desirable formation of gel structure. Furthermore, the effects of activator type and dosage, raw materials' composition and curing conditions on reaction kinetics, gel characters, mechanical properties, and durability issues were also intensively investigated (Bernal et al., 2011, 2012a; Chi and Huang, 2013; Gao et al., 2015; Kumar et al., 2010; Puertas et al., 2000). The recent progress in understanding the blended system provides a solid basis for further investigations, and those modified properties demonstrate a promising future for the application of AAMs.

As reviewed in Section 4.4.1, silica-rich materials such as micro- and nanosilica have been utilized in PC-based systems as an effective pozzolanic binder for the improvement of mechanical properties and microstructure (Pacheco-Torgal et al., 2013). Meanwhile, attention has also been paid to apply nanosilica in alkali-activated systems. Živica et al. (Rouseková et al., 1997; Živica, 2004, 2006) have used chemically modified silica fume combined with NaOH as an alkaline activator in the production of high-performance activated BFS (blast-furnace slag) binders, identifying a highly densified structure and enhanced mechanical strength compared to binders produced using commercial sodium silicate solutions. Bernal et al. (2012a,b) also identified improved mechanical strengths, along with comparable structural features, in BFS/metakaolin binders activated by chemically modified silica fume. Alternative sources of silicon such as rice husk ash (Bernal et al., 2012b) and nanosilica (Rodríguez et al., 2013) have also been assessed as substitute silica sources in these alternative activators, showing that the combination of alkalis with highly amorphous Si-containing precursors can successfully be used as activators in the production of AAMs. Tanakorn et al. (Phoongernkham et al., 2014) studied the effect of nanosilica on fly ash-based geopolymers and showed that increasing the nanosilica content results in the decrease of setting time and the increase of mechanical properties. Microstructure analysis revealed that a nanosilica content up to 2% increases the reaction product and densifies the matrix, but higher nanosilica contents show negative effects. Gao et al. (2013) applied nanosilica up to 3% by mass in metakaolin-based geopolymers. Increased reaction products due to nanosilica addition were observed, and an optimum nanosilica content of 1% in terms of strength and porosity was reported. Adak et al. (2014) investigated the effect of nanosilica on strength and durability of fly ash-based geopolymers; the used nanosilica content was up to 10% by mass. The results showed that a nanosilica content of 6% exhibits appreciable mechanical properties under ambient temperature, as well as less water absorption and reduced charge passed in rapid chloride ion penetration test.

Gao et al. (2015) further investigated the mechanism and performance concerning the effect of nanosilica on the blended alkaline system (Na_2O–CaO–Al_2O_3–SiO_2). The following findings were reported: (1) The incorporation of nanosilica significantly decreases the slump flow of paste samples due to its high surface area and also mixes with a lower slag/fly ash ratio contributing to a better flowability; (2) the nanosilica replacement slightly retards the reaction process at early ages and also leads to a slight reduction of the peak intensity in the main reaction stage; (3) gel character analyses conducted by FTIR (Fourier transform infrared spectroscopy) and TG/DSC (thermogravimetric analysis/differential scanning calorimetry) show that the main reaction product is a C-A-S-H type gel with chain structure, and the addition of nanosilica slightly increases the chemically bound water content; (4) a nanosilica addition up to 2% benefits the compressive strength at typical curing ages of 3, 7, and 28 days, while a further higher nanosilica content shows negative effects; and (5) the porosity decreases with the increasing nanosilica content, indicating the positive effect of nanosilica on pore structure refinement.

4.4.4 Advanced construction materials using photocatalysis

Air pollution caused by road traffic and industry is one of the major problems in metropolitan and urban areas. Despite intensifying emission control requirements and the increased installation of emission reduction systems, air pollution and in particular pollution from nitrogen oxides (NO and NO_2, also written as NO_x) from diesel engines is and will continue to be a serious issue in the near future. By far the largest emissions are generated by traffic and industrial flue gases. Indoor air quality (IAQ) has received great attention because of the very important role the indoor environment plays on human comfort and health. NO_x, sulfur dioxide (SO_2), and VOCs, as typical inorganic and organic indoor air pollutants, can be emitted from cooking, combustion, exhaust gases, tobacco smoke, furniture, building materials, even traffic pollutants from outside, and can cause serious health problems such as drowsiness, headache, sore throat, and mental fatigue (Obee and Brown, 1995). So, it is of vital importance to remove these pollutants to improve the IAQ.

Traditional methods of reducing indoor air pollution include controlling pollutant sources, increasing air exchange, and using air purifiers, which have certain disadvantages. Source control is difficult to achieve in many places. Increasing the air exchange may even transport more pollutants from the outdoor environment (Jones, 1999). Common air purifiers often use sorption materials to adsorb gases or odors which only transfer the contaminants to another phase rather than eliminating them, so that additional disposal or handling steps are subsequently required.

Heterogeneous PCO has been studied for several decades and shown as an effective method for water or air purification. The history of the research on the heterogeneous PCO can be traced back to the 1970s, when Honda and Fujishima (1972) observed the water-splitting effect by the electrochemical photocatalysis process at a semiconductor electrode. Interest from both the scientific and industrial communities in heterogeneous photocatalysis has greatly increased, including fields such as self-cleaning, water purification, air purification, etc. Related publications numbered <100 annually before 200 but were already about 1000 in 2010 (Augugliaro et al., 2010). Studies on PCO of NO_x and VOCs have been carried out intensively (Ao and Lee, 2004; Devahasdin et al., 2003; Hüsken et al., 2007; Obee and Brown, 1995; Ollis and Turchi, 1990; Wang et al., 2007b). Since the middle of the 1990s efforts have been made, first in Japan, in large-scale applications of this photocatalytic reaction for air-purifying purposes and self-cleaning applications. The construction industry has provided several products containing photocatalytic materials on a commercial basis since the middle of the 1990s. Examples of these products include window glass and ceramic tiles providing self-cleaning features. Self-cleaning abilities of modified blends of cement were used for the first time in 1998 for the construction of the church "Dives in Misericordia" in Rome.

For the degradation of exhaust gases originating from traffic, a sheetlike application close to the source is desirable. Suitable surfaces for applications which are close to the source of the emission can be found in large illuminated surfaces in the direct road environment. These are, for example, noise barrier walls and the road or sidewalk surfaces itself. The production of the first concrete paving blocks specially designed for the degradation of exhaust gases started in 1997 in Japan. In 2002, investigations about the application of a cement-based asphalt slurry seal were conducted in Italy.

Although PCO is still quite a novel research topic and many efforts are yet to be spent on it, it has already been widely investigated on many materials such as glass, building materials, plastic, textile, metal, paper, etc. (Augugliaro et al., 2010). TiO_2-containing glass is already available commercially, and usually has the characteristics of being photocatalytic, self-cleaning, hydrophilic, as well as even having improved optical properties. Cementitious materials-related products like concrete are also modified by simply mixing with small amounts of TiO_2. It has been demonstrated (Ballari and Brouwers, 2013; Beeldens, 2007) that the outdoor air purification is achieved by the TiO_2-incorporating concrete pavestones, and some patents (Kimura et al., 2001; Murata et al., 1999) have been granted regarding these photocatalytic products. In addition, photocatalytic concrete has another big advantage of maintaining a clean surface, that is, the self-cleaning property. The application of recycled glass to photocatalytic concrete is a novel idea to address on one hand the sustainability and on the other hand to improve the PCO efficiency, since more light can reach the photocatalyst due to the transparency of the glass (Chen and Poon, 2009). Ceramic tiles, another widely used building material, have also been introduced into the photocatalytic world (Augugliaro et al., 2010).

Recent research focus has been gradually moved to IAQ research as people in general spend over 80% of their time indoors. However, a traditional photocatalyst, for example, TiO_2, can only be activated by UV light which is only 0.001–0.05 W/m^2 in indoor illumination (Kuo et al., 2007). To use TiO_2 as a photocatalyst in the visible light range, it is necessary to enlarge the photoabsorption of TiO_2 into the visible light range (wavelength of 400–700 nm). The usual method is to modify TiO_2 by creating intraband gap states that are close to the conduction or valence band edges and adsorb visible light at subband gap energies of <3.2 eV by using methods such as the sol–gel method and doping metal or nonmetal materials onto the surface of TiO_2 (Anpo, 1997; Asahi et al., 2001; Choi et al., 1994; Kraeutler and Bard, 1978; Sano et al., 2003; Yamashita et al., 1999). Various air-purifying products for indoor air improvement have been researched, and some are already commercially available (Lorencik et al., 2015; Luévano-Hipólito et al., 2014; Pelzers et al., 2014; Wang et al., 2007a; Yu and Brouwers, 2009).

4.4.5 Phase change materials for energy storage

Phase change materials (PCMs) are substances with a relatively high enthalpy of fusion being able to exchange a great amount of heat during the phase change from solid to liquid or vice versa. The melting process absorbs energy (endothermic), and the solidifying process releases energy (exothermic). Paraffin C13-C14 is an example of this type of material, having a melting point between 22°C and 24°C with a heat of fusion of 189 kJ/kg (Zhou et al., 2012). These materials are very attractive for the building sector because they can be used to store or release energy during the peak temperatures of the day, stabilizing the temperature inside buildings and improving the thermal comfort. The PCMs can be classified into three groups: organic, inorganic, and eutectics, each presenting different benefits and drawbacks. Two interesting reviews about PCMs in building materials were given by Zhou and Baetens (Baetens et al., 2010; Zhou et al., 2012). Some drawbacks of these materials are the flammability, instability, large volume change during the phase change, low TC and corrosion. These drawbacks can be removed or reduced by using an outer shell that confines these materials inside an inert package. Nowadays, most research is being done on the micro- and nanoencapsulation of PCMs inside of inert polymer shells (Tumirah et al., 2014; Tyagi et al., 2011; Zhang et al., 2012).

4.4.6 Batteries

Elements that can store energy are of crucial importance in our society. Batteries are one of these storage systems and can be found in portable electronic devices (eg, smartphones, tablets, laptops, power tools), electronic vehicles and renewable energy elements. Nanostructured materials, which are materials that do not have sizes in the nanoscale but have the typical specific surface area and porosity of nanomaterials, can improve the performance of lithium ion batteries in a couple of ways: faster kinetics, and as a consequence, shorter charging times as well as better cycle stability (Lee and Cho, 2011; Wu et al., 2012). Besides lithium batteries, electrochemical capacitors are another energy storage system, which has been extensively studied in recent years (Simon and Gogotsi, 2008; Wang et al., 2012; Yu et al., 2013). An example of an electrochemical capacitor is a hybrid nanostructured material where carbon-based materials are combined with metal oxides (Yu et al., 2013).

4.4.7 Solar panels

Solar energy has developed tremendously in recent decades due to the demand for clean and sustainable energy. Solar panels have been incorporated into buildings to generate electricity (photovoltaic cells and/or solar cells), which is accumulated in batteries, or hot water (solar thermal cell) (Parida et al., 2011). Photovoltaic cells can be classified into first-, second- and third-generation cells. The first-generation cells are made up of crystalline silicon. In the second generation, thin films made of silicon, CdTe or CIGS are deposited on a supporting substrate. The third generation is still under research, which focuses mainly on quantum dots and semiconductor nanocrystals. These two nanomaterials take advantage of the multiple excitation phenomenon, which produces higher electron current and therefore more electricity (Kamat, 2008; Semonin et al., 2011).

4.4.8 Future trends

Due to the unique properties of nanomaterials and nanotechnology, the application of nanotechnology in sustainable construction is currently and will continue to be a very hot topic. The following several main research- and application-related streams will be the future trends:

(1) Understanding of the interaction between the nanomaterials and implanted base materials. This can be achieved by modeling methods, in order to systematically describe and understand the experimental findings from a mechanism point of view. For instance, a deeper understanding of the C-S-H modification by nanosilica is needed.
(2) Better understanding of the environmental and health effects nanotechnology brings to human beings. This triggers researches on certain topics such as the toxicity of nanomaterials, safety issues during manufacture, and disposal manner after demolishing of the structures where nanomaterials are applied.
(3) Experimental facilities and approaches to research. In order to deal with nanomaterials, new analytical instruments for detecting, imaging, analyzing, and evaluating concerning nanoscale synthesis, nanoscale assembly and nanoscale fabrication need to be developed.
(4) Proposal of new standards and improvement of certain current standards: (1) for assessment of the novel performance induced by nanotechnology and (2) during utilization.

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Sustainability of glass in construction

5

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5.1 Introduction

Glass is one of the most favoured materials with widespread applications, such as a building construction material, use in containers and vessels, as windows in the automobile industry, in nanotechnology applications such as optical fibres, etc. Many qualities make glass attractive, as it is transparent, chemically inert, environmentally friendly, sustainable, strong, easily available and relatively cheap. There is no other widely available material which possesses these qualities. In the construction industry, glass has traditionally been used as window panes in buildings, but the use of glass as a main building material has become increasingly popular during the past 25 years (IStructE, 2014). The recent advances in glass technology and the architectural, sustainability and environmental considerations promote the use of glass in buildings. It is hard to envisage any modern building without glass windows/façades. The use of glass to build well-lit and spacious buildings has become more prominent. A modern glass building structure is shown in Fig. 5.1.

Fig. 5.1 Apple Store on Fifth Avenue, New York.

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5.2 Silica glass

The main compound of glass is silica (SiO_2), which is the primary constituent of sand. Natural glass, which is existing for millions of years, formed when certain types of rocks melted as a result of high-temperature phenomena such as volcanic eruptions and lightning, and then cooled and solidified rapidly (Le Bourhis, 2008). Stone Age people may have used natural glass as tools owing to its high strength and sharpness. The oldest use of manufactured glass may have happened around 3500 BC in Egypt where coloured glass was used as jewellery and as vessels to store liquids (IStructE, 2014). Manufactured glass contains considerable amounts of various metal oxides, mainly soda (Na_2O) and lime (CaO), in addition to the main constituent, silica. Therefore, it is known as 'soda–lime–silica glass' or 'soda–lime glass'. Soda–lime glass is the most widely used silica-glass type in the world. Typically, soda–lime glass contains 69–74% silica, 5–14% lime, 10–16% soda and other minor ingredients such as magnesia (MgO) and alumina (Al_2O_3) (Haldimann et al., 2008). The function of soda is to lower the melting point of soda–lime glass to a value between 400 °C and 600°C from that of 1723°C of silica (Le Bourhis, 2008). This low melting point enables the bulk production of soda–lime glass. Glass containing only silica and soda will have poor durability, and it is often water soluble; the addition of lime makes glass more durable. Pure silica glass is still used in special applications, for instance, in windows of spacecraft where glass is exposed to temperatures up to 1200°C.

Other varieties of silica glass which possess specific properties include borosilicate glass, lead-oxide glass (crystal glass) and aluminosilicate glass. In borosilicate glass, boric oxide (B_2O_3) is used instead of soda in soda–lime glass. Borosilicate glass has a high resistance to temperature changes and therefore is commonly used in household cookware. Lead-oxide glass contains 18–40% of lead oxide (PbO) instead of soda in soda–lime glass (IStructE, 2014). Owing to its high density, lead-oxide glass has a high refractive index, and as a result, lead-oxide glass possesses attractive optical properties. Lead-oxide glass is used to produce perfectly clear and flawless objects such as glassware. Aluminosilicate glass contains about 20% alumina and small amounts of lime, magnesia and boric oxide, but only a very small amount of soda. Aluminosilicate glass has the ability to withstand high temperatures and is typically used to manufacture fibreglass and glass fibre–reinforced polymers.

Soda–lime glass in its flat sheet form is the glass type most used in the construction industry; the discussion in this chapter is limited to soda–lime, flat glass sheets. Readers interested in other types of glass should refer to subject-specific text (eg, Le Bourhis, 2008).

5.3 Production of soda–lime–silica flat glass sheets

The float glass process, which was originally developed by Pilkington Brothers in 1959 (Haldimann et al., 2008), is the most common manufacturing process of flat glass sheets. More than 80–85% of the global production of float glass is used in the construction industry (Glass for Europe, 2015a). In the float glass process, the ingredients (silica, lime, soda, etc.) are first blended with cullet (recycled broken glass) and then heated in a furnace to around 1600°C to form molten glass. The molten glass is then fed onto the top of a molten tin bath. A flat glass ribbon of uniform thickness is produced by flowing molten glass on the tin bath under controlled heating. At the end of the tin bath, the glass is slowly cooled down, and is then fed into the annealing lehr for further controlled gradual cooling down. The thickness of the glass ribbon is controlled by changing the speed at which the glass ribbon moves into the annealing lehr. Typically, glass is cut to large sheets of 3 m × 6 m. Flat glass sheets of thickness 2–22 mm are commercially produced from this process. Usually, glass of thickness up to 12 mm is available in the market, and much thicker glass may be available on request. A schematic diagram of the production process of float glass is shown in Fig. 5.2.

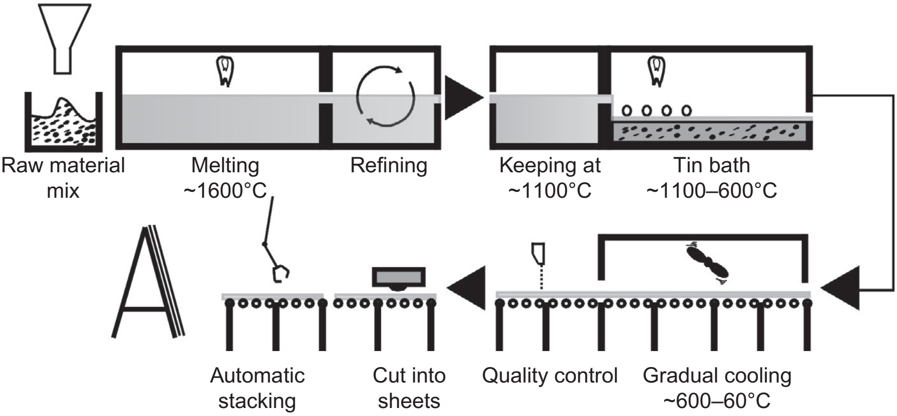

Fig. 5.2 A schematic diagram of the production process of float glass.

Once manufactured float glass, which is also known as annealed glass, is sometimes processed further to produce tempered glass and/or laminated glass. Tempered glass is also known as toughened glass and is stronger than float glass. The laminated glass has enhanced postbreakage performances, safety on impact, improved fire resistance and special properties such as noise control. Details of the manufacturing methods and the mechanical characteristics of tempered and laminated glass are discussed in Section 5.9.

5.4 Properties having influence on choice of glass as a construction material

A unique combination of fascinating physical, optical, chemical and thermal properties makes glass the most preferred construction material in modern buildings. The appropriate use of glass windows, doors, roofs, staircases, partitions, etc. makes buildings bright, airy, energy efficient, and it also enhances the comfort of the occupants.

5.4.1 Physical and optical properties

The most striking feature that contributes to the widespread use of glass in buildings is its transparency to visible light. Owing to the absence of internal subdivisions such as grain boundaries in the microstructure, glass does not scatter light, and as a result, it is transparent. Glass also has smooth surfaces, since during the formation the molecules of the supercooled liquid are not forced to dispose in rigid crystal geometries and can follow surface tension (Haldimann et al., 2008). The recent developments of high-tech glass products expand the range of applications of glass beyond the merely decorative, to functional and structural roles. Light, comfort, well-being, style, safety, security and sustainability are among the benefits that can be achieved from the appropriate use of modern glass products in buildings.

5.4.2 Chemical and thermal properties

One of the key properties of glass is its chemical inertness and general resistance to exposed environment. Glass is one of the most durable materials used in the construction industry. The chemical inertness is attributed to its microstructure: an irregular network of silicon and oxygen atoms with alkaline parts in between. Glass is also an electric insulator, since there are no charged particles such as free electrons in metals or ions in an electrolyte fluid that can move, creating an electric current. The thermal expansion coefficient of soda–lime glass is $8–9\times10^{-6}\,K^{-1}$, and this is of similar magnitude to that of the two most widely used construction materials, concrete ($12\times10^{-6}\,K^{-1}$) and steel ($11–13\times10^{-6}\,K^{-1}$). The specific heat of glass (the amount of heat required to raise the temperature of unit mass of glass by 1 K) is $\sim0.8\,J\,g^{-1}\,K^{-1}$. The thermal conductivity of glass (the amount of heat transmitted through a unit thickness—in a direction normal to a surface of unit area—due to unit temperature gradient under steady state conditions) is $\sim1\,W\,m^{-1}\,K^{-1}$. The relatively low specific heat, high thermal conductivity and the use of less volumes of the material mean glass members have lower thermal mass than that of equivalent concrete/steel/masonry structures.

5.4.3 Stress corrosion cracking

Despite its well-known chemical inertness characteristics, glass is susceptible to stress corrosion cracking: Small flaws grow slowly when exposed to crack-opening stresses in the presence of water or water vapour. This phenomenon is also known as ‘static fatigue’ or ‘slow crack growth’. Typically, under moderate tensile stresses, subcritical cracks in silica glass can propagate at velocities of 10^{-12} to $10^{-5}\,ms^{-1}$ (Lechenault et al., 2011). The mechanism of stress corrosion cracking is complex, and despite the problem having been studied since the 1960s (Wiederhorn, 1968, 1969), the process is not yet fully understood. One of the most acknowledged mechanisms was proposed by Michalske and Bunker (1984): Water molecules break the Si–O bonds located at the crack tip as a result of a hydrolysis reaction. Under modest applied tensile stresses, the crack velocity is governed by the rate of the chemical reaction, which depends on both the degree of humidity and the applied tensile stress. Because of stress corrosion cracking, the strength of glass depends on the rate of the applied loadings. In-depth discussions of stress corrosion cracking in glass may be found elsewhere (eg, Ciccotti, 2009).

5.4.4 Surface coatings

Properties such as visual appearance, optical and thermal properties of float glass may be modified by applying surface coatings. For instance, coatings can regulate certain wavelengths of visible and nonvisible light which are reflected and/or transmitted through glass, and thereby able to control solar energy passing through the glass or to reflect the heat energy back inside the building. The coatings may be applied either offline or online. Offline coatings are applied after the glass is manufactured and cut, usually by dipping glass panes into chemical solutions or by the evaporation of metals onto surfaces in a vacuum. Coatings which give different colours, reflectivities and thermal properties are applied in this way. Online coatings are applied whilst the glass is hot and still in the lehr, and as a result, they form a strong bond to the glass and are usually more durable than offline coatings. Solar control and low emissivity (low-e) coatings are mostly applied online. Multifunctional coatings are used in modern high-tech glass products, the coatings do not adversely affect the desired properties of the original glass. The coatings are durable and usually last as long as the glass member. The applications of some of the special surface coatings are discussed in Section 5.6.

5.5 Glass as a construction material

The worldwide increase of CO_2 emissions because of the high consumption of energy is signalling an alarm for society to focus more on low carbon and energy-efficient buildings. There is a need for professionals in the construction industry and also for governments to be well versed in ways to engineer a sustainable built environment. Glass proves to be a very attractive building material, which provides opportunities for the development of innovative, energy-efficient building envelopes. Glass also has invaluable use in renewable solar energy technologies, such as photovoltaic systems and in solar thermal collectors.

5.5.1 Low carbon and sustainable construction

In simple terms, sustainability is the more prudent use of natural resources for the protection of the environment. It is anticipated that the global population will be increased to 9 billion people in 40 years from the current population of about 7.2 billion. This poses great challenges for the development and maintenance of key infrastructures, such as housing, energy, transport, communications, waste and water, whilst offsetting negative economic, social and environmental impacts. Sustainable constructions are required to improve the long-term social and ecological health of key infrastructures.

5.5.1.1 UK construction strategy

The global construction market and the sustainable construction industry are forecast to grow by over 70% and 23% respectively, by 2025 (compared to 2012) as a result of the low carbon regulatory requirements and the greater social demands for greener products (Great Britain, 2013a). A significant business opportunity exists for low carbon construction; it is expected that the requirement for a green society will drive the future construction markets over the next few decades. In recent years, the UK government put forward clear aspirations which provide confidence in businesses and people to invest in sustainable and low carbon construction. For instance, UK's Construction Strategy 2025 explicitly identifies low carbon and sustainable construction together with ‘smart construction and digital design’ and ‘improved trade performance’ as the strategic priorities which underpin sustained growth across the economy and improved quality of life for people (Great Britain, 2013a).

5.5.2 Low carbon, energy-efficient buildings

The UK goal is to achieve 80% reduction in carbon emissions by 2050 (taking 1990 as the baseline year). Although extensive measures towards low carbon footprint have already been undertaken in some industries (eg, electricity generation), the construction industry has not yet made significant progress. Buildings presently account for nearly 45% of total carbon emission (The Royal Academy of Engineering, 2010). The scale of the challenge in reducing the carbon footprint of buildings is immense; the construction industry must adapt to the low carbon paradigm. The design of low carbon, energy-efficient buildings is quite distinct from the designs of traditional 20th-century ‘energy hungry’ buildings; creative designs and rigorous engineering analyses are required. A ‘green’ building should encompass six main features of sustainability: sustainable sites, water efficiency, energy and atmosphere, material and resources, indoor environmental quality and innovation in design (US Green Building Council, 2012).

The carbon footprint of a building is generally the CO_2 equivalent of all greenhouse gases associated with the construction and operation of the building throughout its lifetime (Great Britain, 2013b). The carbon footprint is twofold: (1) Capital carbon (also referred to as embodied carbon) is the total embodied carbon of the materials and the construction process, and (2) operational carbon is the carbon associated with operation and maintenance, for instance, fossil fuel required for lighting, heating and all other day-to-day operations of the building. Reducing the carbon footprint is fundamentally important to long-term global economy, social and environmental sustainability. Although the low carbon paradigm is an enormous challenge, its pursuance stimulates innovation in design and construction where the reductions in capital and operational carbon ensure resource efficiency and economic benefits.

5.6 Applications of glass to engineer reductions in operational carbon

Many buildings constructed in the 20th century are mainly dependent on fossil fuel energy to make them habitable. As the need for sustainable buildings is more pressing than ever, energy efficiency is the principal driver of modern buildings. Energy-efficient buildings are required to limit the growing gulf between carbon regulations and what is being actually delivered at present. Although zero sum energy buildings may be built by installing renewable energy sources alongside conventional buildings, such designs are expensive and are not sustainable. Glass offers dynamic design solutions which have potential to make buildings energy efficient through the use of daylight and solar gain whilst conserving energy. Glass is the most striking feature in modern building designs.

5.6.1 Features and benefits of glass in buildings

Energy required for heating, cooling, lighting and ventilating of conventional buildings contributes to a major portion of the total carbon footprint. By combining the knowledge of 'building engineering physics' (ie, exploitation of natural science that relates to the performance of buildings and their indoor and outdoor designs) together with the creative use of glass in building envelopes, it is possible to reduce the demand for artificial energy.

5.6.1.1 Daylighting

The most striking property of soda–lime glass is its transparency to visible light (wavelength, $\lambda \approx 380$–$750\,\text{nm}$). Glass has a refractive index of 1.5, and the reflection of visual light is about 4% per surface; hence, the transmissivity of one glass sheet (ie, two surfaces) is more than 90% (Haldimann et al., 2008). By applying special surface coatings, the transmissivity of glass can be further improved and such high-transmissivity glass are available in the market. Because of transparency glass, whether a window or a fully glazed façade, can provide invaluable daylighting into buildings. For an example, Fig. 5.3 shows a modern building which is lit well using daylighting. Daylighting is essential for the function of buildings, and it also helps to improve the health and productivity and to regulate the biological clock of the occupants. Proper daylighting

Fig. 5.3 An example of a modern building which is well lit using daylighting.

designs can avoid the need for artificial lighting for a majority of the day/year, and consequently, can lead to 30–50% savings in the total energy bill of certain buildings (Aboulnaga, 2006).

Although daylighting can easily be provided through glass windows/façades, it is necessary to monitor the intensity, distribution, glare, colour rendering, etc., to create stimulating high-quality interior environments. To achieve a good lighting design, many factors must be taken into account: characteristics of glazing and its orientation, solar control elements such as blinds and louvers, the geometry and the space organisation of the building, surface properties of internal partitions, and distance and orientation with respect to windows/glazing. Inefficient designs could lead not only to poor daylighting, but could also adversely affect the comfort and the productivity of the occupants. Daylighting systems should be free from too much solar gain, brightness, glare, nonuniform lighting, etc. Solar control can be achieved by using specially designed solar control glass. It is possible to offset glare interfering with work tasks such as from computer screens by the use of exterior shading, window blinds, reflective louvers, low-transmission glass and optimal placement of windows/glazing. The provision of high windows with a sloped ceiling together with light shelves that redirect light brings daylight deep into interior spaces without overheating and glare. Readers interested in in-depth discussions of daylighting designs should refer to subject-specific text (eg, Baker and Steemers, 2014).

5.6.1.2 Solar control glass

Visual light passes through glass and heats up the interior. The emitted long-wave thermal radiation is unable to escape through glass because it is absorbed by Si–O groups; this causes the greenhouse effect. In winter, the fact that glass allows solar gain as well as light into the building is beneficial, but in summer months, without solar control, it can become uncomfortably hot. Glass building envelopes should be able to ensure maximum comfort and aesthetic appearance whilst minimising energy consumption throughout the year irrespective of the climatic conditions. The solar control glass products available in the market can be used to avoid overheating of buildings whilst still maintaining high levels of daylight when exposed to sun. Solar control glass is required in buildings which have large areas of glass façades such as the modern commercial building shown in Fig. 5.4.

Solar control glass regulates solar radiation by managing reflection, transmittance and absorption. In the past, highly reflective glass was used in windows to control solar gain, but the use of reflective glass also reduces daylight entering buildings and mirrorlike buildings, as a result of reflection of light, are not architecturally pleasing. Modern solar control glass uses tinted/translucent/opaque/patterned coatings or interlayers to regulate the passage of solar radiation. Solar control glass with interlayers that block UV (ultraviolet) light can be used to protect materials which are sensitive to UV (Nitz and Hartwig, 2005). Generally, modern solar control glass products possess multifunctional benefits, such as low-e, thermal insulation, noise/sound control, etc. In-depth discussions of solar-controlling technologies used in modern glass products can be found in subject-specific text (eg, Nitz and Hartwig, 2005; Smith et al., 2002).

Fig. 5.4 Solar control glass is required in buildings which have large areas of glass façades.

5.6.1.3 *Thermally insulated glazing and low-e glass*

Thermal insulation is an issue of great interest in colder countries where energy is required for space heating. Depending on the exposed environment, up to 25% of the heat from residential and commercial buildings may escape through the windows (Jelle et al., 2012). To minimise the environmental impacts and the rising energy bills, it is essential to save energy. The United Kingdom and other governments regulate minimum requirements for energy efficiency. For instance, the 'Green Deal' introduced in the United Kingdom encourages homeowners to enhance energy efficiency through the provision of an upfront finance to undertake energy improvement measures, with repayments over time offset by savings on energy bills (Great Britain, 2010). Thermally insulated glass can stabilise the internal temperature, and consequently be able to reduce the energy need for heating and cooling. Improvements in thermal insulation also allow the incorporation of larger areas of glazing for daylighting and solar gain.

The recent advances in thermally insulated glazing technology include insulating glass units (IGUs) and low-e glass along with improvements in frames and spacing designs (Sadineni et al., 2011). In an IGU unit, two or more glass panes enclose a sealed air space whilst the whole unit is assembled by a secondary edge seal, usually silicone. IGUs have low heat transfer coefficients (U-values) of about $1\,\mathrm{W\,m^{-2}\,K^{-1}}$ and about

$0.7\,W\,m^{-2}\,K^{-1}$ for double-glazed and triple-glazed units, respectively; the values are significantly lower than the U value of $5.8\,W\,m^{-2}\,K^{-1}$ of conventional single glazing (Haldimann et al., 2008). Low-e glass has an invisible coating (usually tin oxide or a silver-based coating) (Hammarberg and Roos, 2003), which regulates wavelengths of energy, thereby reducing the heat transfer and reflecting the heat back into the interior. Low-e glass is more suitable for rooms/buildings which have high proportions of windows/glass doors. Manufacturers of low-e glass products (Pilkington, 2015b; Dupont, 2015) claim savings of up to 75% compared to single glazing. In-depth discussions of thermal-insulating technologies used in modern glass products can be found in subject-specific text (eg, Sadineni et al., 2011).

5.6.1.4 Noise control glass

Since the windows are the primary path through which noise enters a building, it is necessary to have sound insulation glass in noisier environments, such as those close to airports, highways, cities, etc. Propagation of sound may be retarded by either reflecting the noise back towards the source, or by absorbing the energy within the glass. Resin-based interlayers, those bonded between the glass sheets of laminated glass, are used to reduce the propagation of sound through glass windows. Damping effects of the interlayer retard the vibrations, thereby suppressing the acoustic noise. The mass of the glass also has a significant effect on the sound attenuation; thick glass laminates usually possess satisfactory acoustic properties. Noise control, laminated glass is available in combination with other special properties such as solar control and low-e. In-depth discussions of noise-controlling technologies used in modern glass products can be found in subject-specific text (eg, Zhu et al., 2004).

5.6.1.5 Vibration control glass

As in noise control glass, dissipation of energy is required to control vibrations. Viscoelastic materials are widely used in many industries to damp vibrations; laminated glass with interlayers, which possess damping properties, are widely used in glass members subject to vibration-induced loadings, for instance, in floor plates and treads for staircases in buildings (Haldimann et al., 2008). A more in-depth discussion of contemporary glass products is beyond the scope of this chapter. Interested readers should refer to detailed text on the subject (eg, Koutsawa and Daya, 2007).

5.6.1.6 Self-cleaning glass

Buildup of dirt (soiling) and the subsequent decay of optical properties is one of the main problems encountered in high-rise buildings with large glazing. Fig. 5.5 shows a glass window which has lost optical properties as a result of the buildup of dirt. Due to the cost and the challenge associated with cleaning of dirty glass, the use of self-cleaning glazing has become popular in recent years. A special coating (usually nanostructured TiO_2), which has an innovative dual action, is used in modern self-cleaning glass products. Once exposed to sunlight (UV radiation), the coating chemically reacts with oxygen and the water molecules present in the atmosphere, and subsequently

Fig. 5.5 A glass window which has lost optical properties due to the buildup of dirt.

breaks down the organic dirt deposits, for example, bird droppings (Haldimann et al., 2008; Chabasa et al., 2008). The rainwater then easily washes away the loosened particles. Self-cleaning glass also functions well in prolonged dry spells and in areas protected from direct rainfall; it is only necessary to wash down with water. Similar to other special coatings, self-cleaning glass can be combined with other properties such as solar control. An in-depth discussion of modern self-cleaning glass technologies can be found in subject-specific text (eg, Chabasa et al., 2008).

5.6.1.7 Fire-resistant glass

The resistance of glass to high temperatures is low, and glass transmits heat rapidly. Float glass usually breaks due to thermal shock when the temperature difference is about 40°C, whereas toughened glass can withstand temperature differences up to 200°C (IStructE, 2014). Glass also starts to soften and loses stiffness at temperatures above the glass transition temperature (~500°C). Special fire-resistant glass is required to achieve satisfactory performances against fire. Borosilicate glass has relatively high resistance to thermal shocks as opposed to soda–lime glass, since its thermal expansion coefficient ($9 \times 10^{-6}\,K^{-1}$) is higher than that of soda–silica glass ($6 \times 10^{-6}\,K^{-1}$). The modern fire-resistant glass products use laminated glass with intumescent interlayers (Haldimann et al., 2008). When one side of the laminate is exposed to fire, the interlayer expands into an insulating foam after absorbing heat from the fire, and subsequently protects the second glass sheet (Haldimann et al., 2008). The modern glass products have potential to withstand moderate fires expected in residential/commercial buildings for up to 3 hours (Pilkington, 2015a). Old technologies of fire-resistant glass include laminated glass with a wire mesh (see Fig. 5.6), in which the wire mesh keeps glass in place after it cracks. Although the application of a wire mesh has potential to improve fire resistance, it weakens the glass due to the surface flaws induced by the wire mesh.

Fig. 5.6 Wire mesh can improve fire resistance, but the surface flaws caused by the wire mesh degrade the strength of the glass.

5.7 Use of glass in low energy, passive house buildings

At present, a significant portion of the total production of electricity/gas energy is consumed by buildings, especially in developed countries. For instance, about 40% of the total US primary energy is consumed by buildings (United States, 2009). There is an imperative need for energy efficiency in buildings. Energy efficiency is one of the key attributes of a green building. In recent years, there has been a radical overhaul in the approach for building designs where reductions in the demand for artificial energy required for lighting, cooling/heating, ventilation, etc., are intended. The recent developments in energy-saving technologies are mainly twofold: (1) active technologies, such as heat pumps coupled with air/groundwater heat sources, solar thermal collectors, renewable energy sources such as solar photovoltaic panels and wind power, etc., and (2) passive technologies, which include increased insulation, efficient use of daylight and solar gain, heat recovery from ventilation air and/or wastewater, etc. (Sartori and Hestnes, 2007).

The use of active technologies such as renewable energy sources is expensive, and the solutions may also increase the embodied carbon of the buildings. On the other hand, passive technology-based building designs (eg, passive houses), in which the design exploits passive technologies to diminish the energy demand, can significantly reduce the total energy requirement without increasing the embodied carbon or the total construction cost. Passive house systems outperform conventional buildings in terms of living conditions and energy efficiency due to heat recovery, good thermal insulation and the overall optimisation of the buildings (Sartori and Hestnes, 2007). Some renewable energy sources may also be used in these buildings to generate the small energy demand, and consequently, to fulfil zero sum total energy.

Development of passive house buildings is a win–win situation due to the potential reductions in the energy demand and the carbon footprint. The recent innovative advances in glass products (Section 5.6) mean glass has become the most vital building material in low energy, passive house buildings. As described previously, the appropriate use of glass can reduce heat loss but allows solar gain to heat buildings. Proper use of solar control glass whilst maintaining the transmittance of daylight can eliminate the need of artificial air-conditioning systems. In a properly designed passive house building, the internal temperature can be kept between 20°C and 26°C and the indoor relative humidity between 30% and 60%, ensuring the building is comfortable in all seasons. Passive house buildings are now constructed all around the world; significant energy savings and high levels of occupant satisfaction are already being noted. In-depth discussions of passive house designs can be found in subject-specific texts (eg, Sartori and Hestnes, 2007; Baetens et al., 2010; Cheung et al., 2005; Pacheco et al., 2012).

5.8 Glass: A sustainable construction material

The applications of glass in buildings to engineer reductions in operational energy have already been discussed in this chapter. The embodied energy/carbon impact of glass and the sustainability of glass as a construction material, including reuse and recycling, are discussed in the following sections.

5.8.1 Embodied energy and carbon

The embodied energy of a building is the energy consumed by all the materials and the processes associated with the construction of the building—from the mining and processing of natural resources to manufacturing, transport and product delivery (Sattary and Thorpe, 2012). Similarly, the embodied carbon of a building is the total carbon associated with all the materials and processes used over the total life cycle of the building. The annual use of over 400 million tonnes of materials in the UK construction industry (Langdon, 2009) indicates the high embodied energy/carbon impact of the industry. If the United Kingdom is to achieve the ambitious target of 80% reductions in carbon emissions by 2050, greater reductions in the total embodied carbon of buildings are required. This can be achieved by: (1) reducing the amount of materials used and minimising the waste, (2) lessening the use of impact materials and energy-intensive manufacturing methods, and (3) using good environmental management methods including reuse and recycling of the materials (Waste & Resources Action Programme, 2015).

5.8.2 Embodied energy and carbon of common construction materials

The embodied energy/carbon of buildings has been somewhat neglected in government regulations as the current regulations mostly focus on the reduction of operational energy/carbon. According to Rawlinson and Weight (2007), the embodied energy of a typical complex commercial building in the United Kingdom may be equivalent to 30 times its annual operational energy use. Sturgis and Roberts (2010) estimated that embodied carbon can account for up to 45% of the total carbon impact of a building over its life cycle. Although an analysis of embodied energy/carbon is required in order to evaluate the total impact of a given building, a reliable investigation of the embodied energy/carbon is not trivial. For instance, transportation can affect the embodied energy—a material manufactured and used in London has an embodied energy impact different from the same material transported by road to Edinburgh. Recycled materials are sometimes used for the manufacturing of new products, and these products usually have a lesser carbon impact. It is also difficult to take into account the energy required for the maintenance, repair and refurbishment of a building over its life cycle.

Despite the difficulty of accurate analyses of the embodied energy/carbon impact of construction materials over the life cycle of a given building, few methods have been reported in the literature. One such method is the 'University of Bath's inventory of carbon and energy database' (Hammond and Jones, 2006), and this inventory provides an open-access database of energy/carbon impact of over 400 materials (Hammond and Jones, 2008). This database has been employed by various researchers and developers of carbon footprint calculators, including the UK's Environmental Agency's carbon calculator for construction (Hammond and Jones, 2008).

5.8.3 Embodied energy/carbon of glass

Table 5.1 shows a comparison between the embodied energy and carbon values of glass and the two most-used construction materials, concrete and steel.

Although the exact embodied energy/carbon impact of construction materials depends on the actual application, the values quoted in Table 5.1 may be used to investigate the relative impact of glass compared to concrete and steel. Float glass has an embodied energy/carbon of 15/0.232 MJ kg^{-1} which is less than that of steel (24.6/0.466 MJ kg^{-1}), but greater than reinforced concrete (1.39/0.057 MJ kg^{-1}) (Table 5.1). A major portion of the embodied energy/carbon of glass is attributed to the high-temperature production process. Owing to the secondary heating process used in toughened glass, it has greater embodied energy/carbon (23.5/0.346 MJ kg^{-1}) than float glass. It should be noted that, although concrete has a relatively low embodied energy per unit mass, its global impact is greater than that of glass due to the large volume of concrete used in the construction industry (note: reinforced concrete is the most widely used construction material in the world with an estimated annual consumption of over 12 billion m^3 (~25 gigatonnes) of concrete (Gursel et al., 2014) and ~200 billion kg of steel (World Steel, 2010)). In the United Kingdom, the current average embodied carbon impact of concrete is around 100 kg of CO_2 per tonne (The Concrete Centre, 2015). On the other hand, the mass/volume of glass needed to construct members of buildings is less than that required for an equivalent concrete member. In addition, glass is more durable than steel and concrete, and the use of glass also has potential to reduce the operational energy/carbon impact of buildings. Thus, glass is a more sustainable construction material than concrete and steel, in spite of the embodied energy of glass being greater than that of concrete in unit mass basis.

More in-depth discussions on embodied energy and the sustainability of construction materials can be found elsewhere (Hammond and Jones, 2006, 2008; Khatib, 2009).

Table 5.1 **Embodied energy and carbon values of glass, concrete and steel (Hammond and Jones, 2006)**

Material	Embodied energy (MJ kg^{-1})	Embodied carbon (MJ kg^{-1})
Float glass	15	0.232
Toughened glass	23.5	0.346
Reinforced concrete	1.39	0.057
Steel (bar and rod)	24.6	0.466

5.8.4 Reduction of the impact of embodied energy/carbon of glass

Buildings which are efficient in terms of the amount of materials used are also efficient in terms of embodied energy and carbon. The amount of materials required for construction can be optimised by improving the overall efficiency of the designs, for example, better specification of the design guidelines, optimal structural designs, avoiding overengineering, designs for future use including adaptability and flexibility, etc. It is also important to reduce the waste of the materials, for instance, through the use of skilled workmanship and off-site construction. Another important way to reduce the impact of embodied energy/carbon is to recycle and reuse the materials.

5.8.4.1 Recycling glass

Although glass bottles are usually recycled, glass sheets like those used in buildings are not recycled. This is mainly due to the difficulty of removing the coatings and other materials (eg, adhesives, metals, glass produced by other manufactures, etc.) mixed with waste glass. The low energy savings that can be achieved from recycling is another reason for the lack of recycling of glass. Table 5.2 provides the potential energy savings that can be achieved by recycling glass and a few other materials, aluminium, plastic and cardboard (Sattary and Thorpe, 2012).

Table 5.2 **Potential energy savings by recycling glass, aluminium, plastic and cardboard (Sattary and Thorpe, 2012)**

Material	Energy required to produce from virgin material (MJ kg^{-1})	Energy savings by using recycled materials (%)
Float glass	15	5
Aluminium	250	95
Plastic	98	88
Cardboard	26.5	24

Recycling of glass can save only up to 5% of the energy required in the original production using the raw materials (Table 5.2). This is a very low energy saving compared to that can be achieved from recycling aluminium and plastic, where the potential energy savings are 95% and 88%, respectively. During recycling of glass, the cleaned old glass is crushed into small glass pieces and mixes with the original raw materials (silica, lime, soda, etc.). The mix is then heated and annealed in the same way as the production using pure raw materials (see Fig. 5.2). Due to the high energy required for this process, the energy saving that can be achieved by recycling is limited to 5%. Although recycling of waste glass is not particularly appealing, waste glass can be reused in many ways.

5.8.4.2 Reuse of glass

In Europe, each year over 1.2 million tonnes of waste are generated by the demolition of and renovation of buildings in which the waste glass is only about 0.6% (Glass for Europe, 2015b). Glass manufacturers usually do not recycle most of the waste glass, and therefore the reuse of the material is important. Since glass is a hard, relatively inert material, it naturally lends itself to use as aggregate in concrete. Owing to the different colours and glistening when sunlight falls on glass, concrete with glass aggregate possesses excellent aesthetics (see Fig. 5.7). However, glass aggregate in concrete has durability issues due to the alkali-silica reaction. Research investigations, however, showed that the alkali-silica reaction may not happen if the waste glass is finely ground, usually smaller than 75–100 µm size (Corinaldesi et al., 2005).

Another successful application of waste glass is its use as an alternative aggregate in bituminous materials in road construction, where the term *glassphalt* is being used (Khatib, 2009). The amount of glassphalt replacement of conventional aggregate ranges up to 50%, but more commonly about 10% is used (Khatib, 2009). Fig. 5.8 shows a close-up of the surface of a road constructed using glassphalt. Waste glass, after being crushed to a size specified by the end-user, can also be used as glass beads for reflective paint, as pipe cushion for French and storm drain systems, and as an

Fig. 5.7 Use of glass aggregate in concrete.

Fig. 5.8 Surface of a road constructed using glassphalt.

abrasive (such as sandblasting grit) (Khatib, 2009). Other useful applications of fine waste glass are in ashtrays, filter media for swimming pools, golf course sand traps, aquarium sand, etc. (City and County of Honolulu, 2005). The reuse of waste glass is greener than recycling, and the availability of many reuse applications means almost all waste glass is reusable.

5.9 Glass as a structural material

The recent architectural and technological developments in the use of glass in building envelopes pose the challenge for structural engineers to design large areas of glass panels, roofs, floors, staircases and partitions. All these glass members will have a structural role compared to small glass panes used in traditional four-edge supported windows that engineers have been familiar with for centuries. Since glass is a brittle material, its structural behaviour is significantly different from that of more familiar materials such as steel and reinforced concrete. The structural designs of glass must take into account the fundamental material behaviour.

5.9.1 *Mechanical properties of glass*

Glass is a perfectly linear-elastic and isotopic material. The typical values of Young's modulus (E), shear modulus (G), Poisson's ratio (υ) and density (ρ) of glass are given in Table 5.3.

5.9.2 *Strength of glass*

Since glass is a brittle material, its tensile strength depends on the inevitably present surface flaws. The geometry and the distribution of surface flaws are unknowable, and therefore the prediction of the strength of glass is challenging. The analysis of

Table 5.3 **Basic mechanical properties of glass (BS EN 16612, 2013)**

Mechanical property	Value
Young's modulus, E	70 GPa
Shear modulus, G	28.7 GPa
Poisson's ratio, υ	0.22
Density, ρ	2500 kg m^{-3}

molecular forces indicates the tensile strength of glass is as high as 32 GPa (Haldimann et al., 2008), but since the surface flaws cause fracture, the actual tensile strength of float glass is in the range of 20–45 MPa (IStructE, 2014). The strength of glass also depends on the duration and the spatial distribution of the applied loads. The compressive strength of glass is much greater than the tensile strength. However, the compressive strength is not relevant in practical structural designs as the compression members will prematurely fail due to buckling or due to the tensile stresses developed owing to the Poisson's ratio effects. A comprehensive analytical or numerical method to predict the strength of glass structural members is lacking. Consequently, the use of glass as a structural material is not being exploited as effectively as it might be.

5.9.2.1 Practical strength of glass

Experimental methods those recommended in design codes to determine the strength of glass include 4-point bending tests and coaxial double ring tests (BS EN 1288-2, 2000). However, the strength values obtained for a number of test specimens made from one single glass sheet usually show a significant scatter. This may be attributed to the different distributions of the surface flaws present in each test specimen. The strength values obtained from an experiment may not align with any accepted probability distribution; Weibull probability density functions are usually proposed in the design codes, for example, BS EN 12600 (2002). The current design codes provide typical strength values of different types of commercially available glass products. For instance, Table 5.4 shows the characteristic bending strength values recommended in the recently published IStructE structural glass design guidelines (Haldimann et al., 2008) for float glass, heat-strengthened and fully toughened glass. However, due to the differences in the manufacturing processes used by different manufacturers, the quoted strength values must be considered cautiously. In-depth discussions on investigation of strength of glass under different loading conditions can be found in subject-specific text (eg, Haldimann et al., 2008).

5.9.3 Glass in load-bearing structural members

Although float glass is the most readily available glass type, it is rarely used in load-bearing structural members owing to its low tensile strength and the brittle failure behaviour. Toughened and laminated glass are mostly used in load-bearing glass structural members.

Table 5.4 **Characteristic bending strength of different glass types (Haldimann et al., 2008)**

Glass type	Characteristic bending strength (MPa)
Float (annealed)	45
Heat strengthened	70
Fully toughened	120

5.9.3.1 Toughened glass

During the tempering (toughening) process, float glass is heated to about 620–675°C in a furnace and then quenched by jets of air. When cooled the glass surface solidifies first whilst the interior remains hot. When the interior cools the thermal shrinkage is resisted by rigid solid surface, and as a result a residual stress field with tensile stresses in the mid-thickness region is developed. The subsurface tensile stresses are then balanced by compressive residual stresses developed in the surface regions. Typically, surface compressive stress of 80–150 $\mathrm{N\,mm^{-2}}$ is present in fully tempered glass. Owing to the surface compression, toughened glass is about five times stronger than float glass. The depth of the surface compression zone is ~0.2 times the overall thickness of the sheet (Castellini et al., 2015), and the compression layer will often retard the potential propagation of the surface flaws. Fig. 5.9 shows the typical parabolic shape of the residual stress depth profile that exists in fully toughened glass. Generally, modern toughened glass is strong enough to withstand sledgehammer attacks. Toughened glass is about 3–5 times more expensive than float glass (on a unit area basis). The use of tempered glass also adds additional design/construction challenges; penetrations beyond the surface compression layer will lead to fragmentation. In addition, no alterations (eg, cutting, drilling, grinding, etc.) can be done in tempered glass.

5.9.3.2 Heat-strengthened glass

Heat-strengthened glass is produced in the same way as fully toughened glass, but the heated float glass is quenched at a slower cooling rate than that used for full-tempering. As a result, heat-strengthened glass has a low surface precompression of 40–80 MPa compared to that of 80–150 $\mathrm{N\,mm^{-2}}$ in fully tempered glass.

5.9.3.3 Laminated glass

Laminated glass is produced by combining two or more sheets of float/tempered glass with one or more interlayers and is processed by autoclaving at 1400°C and pressure up to 14 bar (Haldimann et al., 2008). Polyvinyl butyral (PVB) is the most common interlayer used in laminated glass. As described in Section 5.6, alternative transparent interlayers may be used in laminated glass to obtain special properties, such as solar control, thermal insulation, fire protection, etc. Laminated glass also provides satisfactory structural behaviours under extreme loading conditions such as explosions.

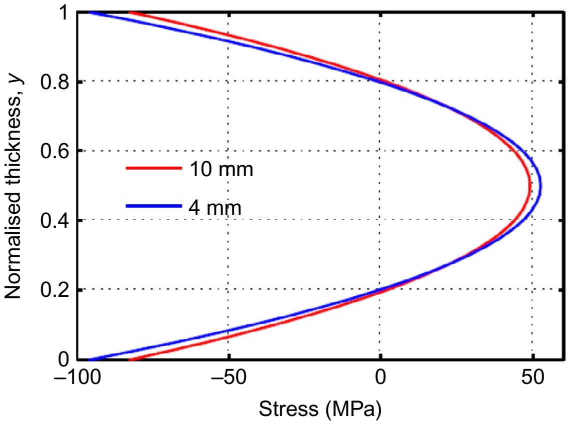

Fig. 5.9 Parabolic residual stress (prestress) depth profiles present in fully toughened glass pieces of thickness 10 and 4 mm, respectively.

One of the main reasons for the use of laminated glass in building envelopes is its safe failure mode compared to that of float glass and tempered glass. Recent developments include high-tech ionoplastic interlayers. According to the manufacturers, laminated glass with ionplastic is lighter and stronger than conventional laminated glass, and can withstand storms, impacts and powerful blasts (Dupont, 2015). It is also worth noting that toughened glass with polycarbonate interlayers is used in bulletproof glass.

5.9.4 Failure mode and postfracture behaviour of glass

The tensile stresses which cause failure of glass specimens may be generated in one of many possible ways, for example, as direct tensile stresses, bending stresses, temperature gradients, due to the initial lack of fit in the structures, etc. Float glass shatters into large pieces of sharp shards as shown in Fig. 5.10A. Shards falling from the top of a building or glass that flows at high speeds are a threat to the occupants of the building. In tempered glass, the residual stress affects the fragmentation. When tempered glass cracks, it releases the residual stress and the cracks progress rapidly, repeatedly bifurcating and causing complete fragmentation of small dice of about 100 mm^2 (Haldimann et al., 2008), as depicted in Fig. 5.10B. Because of the fracture pattern and the high strength of the material, tempered glass is often known as ‘safety glass’. Nevertheless, even small glass dice can cause serious injuries if they flow at high velocities. On the other hand, laminated glass with PVB interlayer is often able to provide an adequate level of postbreakage performance, since the PVB interlayer locks together the broken glass pieces and interacts with the remaining unbroken glass sheet/s (see Fig. 5.10C). The PVB interlayer has a certain degree of tensile strength, and this strength may be utilised during a severe breakage where broken glass pieces can be locked in compression as a result of the arching action.

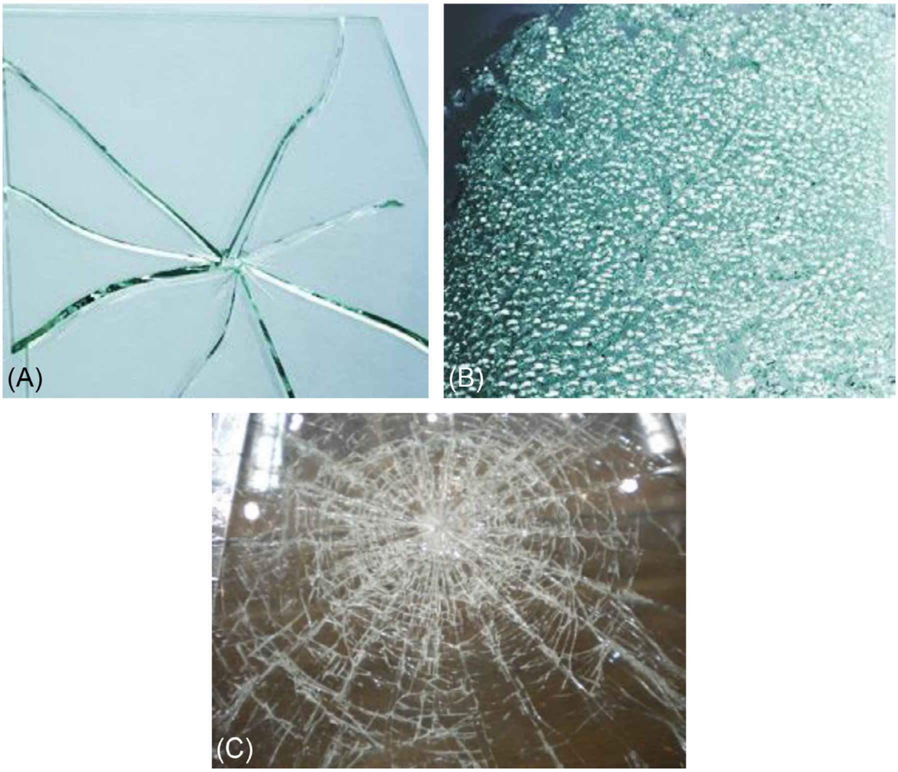

Fig. 5.10 Postfracture behaviour of (A) float, (B) fully tempered and (C) laminated glass.

5.10 Glass structural design criteria

The increasing use of glass as a load-bearing material has led to the development of design standards, technical guidelines and recommendations in recent years, for example, IStructE (2014). A comprehensive overview of all current design standards and design methodologies is beyond the scope of this chapter. A brief overview of the limit state design methodology recommended in the draft Eurocode, BS EN 1288-2 (2000), is described here. Readers interested in in-depth reviews of design codes and design methodologies should refer to subject-specific text (eg, IStructE, 2014).

The ultimate limit state design adopted in BS EN 1288-2 (2000) compares the applied tensile stress with the design glass strength. The applied stresses may be determined using standard methods of structural analysis that are based on loads multiplied by partial factors as defined in current Eurocodes: (1) 'Basis of Structural Design' (BS EN 1990:2002, 2010) and (2) 'Actions on Structures' (BS EN 1991:1-1:2002, 2010). The design strength of glass is determined by applying a series of partial factors to the characteristic strength of glass (the characteristic strength values of different glass types are presented in Table 5.4). These partial factors usually account for: (1) load duration, (2) surface profile of glass, (3) material partial safety factor, (4) partial safety factor for surface prestress (if any), (5) partial safety factor for method of prestressing, etc. The typical values of the material partial factors can be found in BS EN 1288-2 (2000). This design code also recommends the serviceability limit state design guidelines.

5.11 Connections in glass

One of the main challenges that limits the effectiveness of load-bearing large glass structural members is the lack of an effective connection system. The most popular mechanical connection is 'point fixing' (an example is shown in Fig. 5.11) in which a stainless steel bolt is used in a countersunk hole with an intermediate softer liner material (eg, aluminium), which reduces the bearing stresses (Overend et al., 2011). The current design guidelines are based on rules of thumb where a bearing capacity of 1 kN mm^{-1}-width is simply assumed (IStructE, 2014) without explicitly taking into account the surface flaws induced by the drilling process or the high-stress concentration present in the vicinity of the hole. Since glass cannot yield, the development of high-stress concentrations causes failure. The current design guidelines also do not explicitly take into account the residual stress relaxes around the edge of a hole in tempered glass (Achintha and Balan, 2015; Balan and Achintha, 2015).

On the other hand, adhesive-bonded joints are structurally superior to bolted joints for applications ranging from glass–glass to glass hybrid joints as they do not cause stress concentrations or the development of new surface flaws. However, the use of stiff adhesives is a relatively unproven technology for joints in glass. Adhesive joints subject to complex 3D stress and strain states, and these stresses may lead to

Fig. 5.11 Although bolted joints are widely used, they limit the effectiveness of load-bearing glass structural members.

premature failures. A widely accepted experimentally validated design methodology for the analysis of the load response and the failure mechanism of glass–adhesive joint configurations has not been reported in the literature.

5.12 Detailed finite element analysis

The propagation of surface flaws under applied tensile stresses causes failure of glass; therefore, the theory of linear elastic fracture mechanics (LEFMs) provides a reliable method to design glass structures. According to LEFM, a brittle material fails when the stress intensity factor at the critical crack reaches the fracture toughness of the material, which is a known value for a given material. A range of values has been reported in the literature for the fracture toughness of soda–lime glass; the value of 0.75 MPa $m^{0.5}$ is recommended in the design codes (Haldimann et al., 2008). The growth of the critical flaw and the complete lifetime of a glass member may be modelled using LEFM. However, LEFM analyses require the knowledge of the geometric properties of the critical crack, such as its location, crack size and crack shape; in practice, these are unknowable and should be assumed by the analyst/designer. In addition, finite element (FE)-based analyses are required to take into account the effects of the residual stress in a complex geometry. For instance, Fig. 5.12 shows the results obtained from an FE analysis for the residual stress distribution in the vicinity of a hole in a tempered glass plate (note: due to symmetry, only a quarter of the plate is shown in the figure).

5.13 Future trends

As the need for sustainable buildings is more pressing than ever, energy efficiency is the principal driver of new buildings and existing buildings as well. Glass has potential to offer dynamic design solutions that enable buildings to be more energy efficient

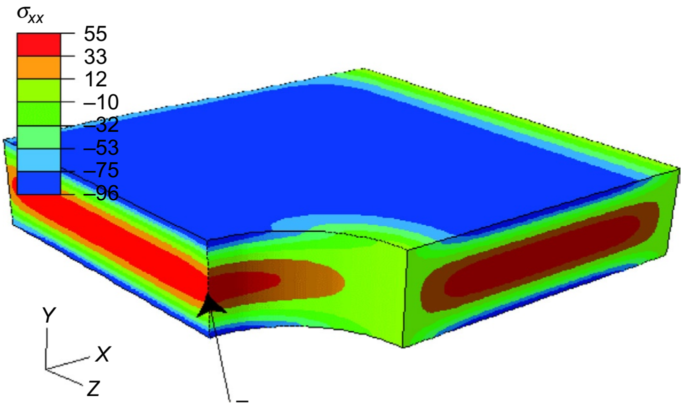

Fig. 5.12 Residual stress distribution in the vicinity of a hole in a fully tempered glass plate.

by making the most of daylight and solar gain whilst protecting the environment and conserving energy. Future trends in the use of glass in buildings are twofold:

(1) The use of smart and truly responsive glass façades in which the properties change to actively control solar gain, daylight, glare and thermal emissions in passive house buildings

Passive technology-based building designs can significantly lower the total energy demand without increasing the embodied carbon or the total construction costs. The use of passive or self-adjustable glass products which control light transmittance by responding to the amount of UV radiation in the exposed daylight will be able to regulate the solar gain. The coatings/interlayers which contain chemicals sensitive to temperature can be used to control thermal properties of glass façades, and thereby to maintain satisfactory indoor environments. Photovoltaic glass, which consists of laminated glass with integrated solar cells, may be used to supply the moderate energy demand of passive house buildings.

(2) The use of glass and glass hybrid systems as load-bearing structural members

Structural engineers will be required to design new large areas of glass panels, roofs, floors, staircases and partitions which all have load-bearing structural roles. FE analysis-based designs may be used to incorporate the effect of residual stress and also to predict the ultimate strength and the lifetime. The use of adhesive-bonded joints, which are structurally superior to bolted joints, will enable the construction of unconventional and large glass structures of complex geometries. The growing use of glass as a load-bearing material means comprehensive design standards, technical guidelines and recommendations will be required.

It is also anticipated that the waste glass from construction/building sites may be reused to a great extent. As described in this chapter, waste glass can be reused in a number of ways, such as aggregate replacement in concrete and bituminous mixtures, ground glass as cement replacement, etc.

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Sustainability of metals and alloys in construction

6

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6.1 Introduction

Unlike most manufactured materials, metals have a long and successful history of recycling. Largely due to the significant inherent value of metals, scrap dealers have been an integral part of industrialised society since Georgian times. With the notable exception of gold and the occasional chunk of meteoric iron, metals are essentially as artificial to nature as plastic bags. Within the earth most metals only exist in stable combined states with other elements such as oxygen and sulphur. Iron, as an example, can be found as an oxide such as hematite or as a sulphide as in pyrite, the infamous 'fool's gold'.

Once we acknowledge the artificial nature of metals it is easier to appreciate the problems of oxidation and corrosion, when these essentially unstable materials convert back to the more stable compounds from which they were refined. Corrosion is a natural and normal process, reverting the metals to their lowest energy state. Ferrous or iron-based alloys such as steel can be considered to have a similar life cycle to their masters, we humans. Once 'born' their lifespan is dependent on the appropriateness of the tasks they are given to perform and how well they are treated. Put to good use and properly cared for, alloys can last to 100 years and beyond. Gold remains all but immortal and is treated accordingly.

The remainder of this chapter will address the matter of the birth, life, death and reincarnation (ie, life-cycle assessment) of metals and how this may act as a blueprint for the reuse and recycling of other material types.

6.2 Ferrous alloys

Ferrous alloys, notably steel but initially also cast and wrought iron, represent the most important family of metallic materials. Iron-based materials account for by far the greatest proportion of manufactured metal items. Their popularity is more than just a simple matter of being relatively cheap or plentiful. Iron and steel offer a range of properties that fit well with the requirements of the manufactured world. They are relatively easy to form, are tough and forgiving in service, resist wear and damage and despite the occasionally bad press can be made sufficiently resistant to corrosion to give adequate service. They are and have always been recyclable, with around 75% of feedstock for the production of new alloys coming from scrap.

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The first iron was extracted by smelting iron oxide, much in the same way copper alloys had been produced since the Bronze Age, the difference being the higher temperature required for iron. While copper melts at a little over 1000°C, the melting point of iron is 1538°C but fortunately drops to 1370°C when alloyed with carbon at 1.7% or higher to produce cast iron, the mainstay of the 18th century's industrial revolution in manufactured goods.

6.2.1 Cast iron

With its historical roots in first-century China, cast iron became a key element of the industrial revolution. First with cannon and shot, then with pots and pans, cast iron allowed the mass production of durable goods for trade and conquest.

Early cast irons were strong, hard and brittle. When fractured, the rough surface caused by the flakes of graphite in the structure appears grey, hence grey cast iron. Lower silicon plus faster cooling produces an even stronger structure with more dispersed carbon that appears lighter when fractured, known as white cast iron.

Further developments in composition and heat treatment lead to the production of tougher, less brittle cast irons with some ductility. More recent modifications through the addition of nickel have allowed the production of cast irons with exceptional strength and enhanced high-temperature properties (Elliot, 1988).

6.2.2 Wrought iron

While cast iron is strong, hard and resistant to wear, it is difficult to form other than by casting and can be brittle. Very early on it was recognised that a more malleable material would have its own benefits in many applications, and wrought iron satisfied that need. Wrought iron, literally ‘worked’ iron, has much lower carbon content, typically 0.15%, and has the ability to be hot worked to produce a wide range of functional and decorative objects in the hands of a skilled blacksmith.

Dating back to pre-Roman times, wrought iron was originally made by smelting iron ore with charcoal, a process by which the mixture is taken to just below the melting point of the iron resulting in a spongy mass or ‘bloom’ of iron and slag (a mixture of oxides, silicates and other impurities). Provided the iron does not melt, the carbon content remains low. There remains, however, the problem of the slag. Mechanically working the bloom removes the majority of the slag, leaving behind a relatively soft and malleable iron interspersed with fine slag inclusions producing the familiar fibrous structure of wrought iron.

The process for the manufacture of wrought iron was continually improved from the Middle Ages onward until, by the start of the 20th century, it could be produced quickly and cheaply in a near-continuous industrial process. While the attraction of wrought iron as a decorative material remains, its manufacture started dying out in the 1960s and has now all but disappeared, having been largely replaced by low carbon steel (Kuhn, 1982).

6.2.3 Steel

The universally familiar steel is a simple alloy of iron and carbon. Other alloying elements such as manganese, silicon, copper and tungsten are commonly also present and can improve or modify the characteristics of the steel. A small amount of manganese is added to reduce the embrittling effects of iron sulphide during secondary steelmaking processes and to ensure low-temperature toughness. Low carbon or 'mild' steels have carbon contents of 0.05%–0.3%, so the high carbon pig iron produced by smelting iron ore and coke must be further refined. In this instance, *low carbon* refers to a metallurgical necessity rather than a measure of its sustainability credentials.

The greatest benefit of low carbon steel is its ability to be cut, shaped and joined with relative ease to produce a wide range of products. Where welding is required, the carbon content is kept below around 0.25%. The ductility of mild steel allows the forming of complex shapes and ensures yielding occurs prior to failure, a valuable attribute when trying to avoid sudden and catastrophic failure of component and structures.

The biggest potential drawback to employing simple low carbon steel, whether mild or high yield, is its need to be protected from the effects of moisture and oxygen leading to corrosion. While paints and greases can be used to great effect to prevent this occurring, there has long been an interest in more highly alloyed steels that can impart a degree of corrosion resistance without the need for protective coatings.

6.3 Stainless steel

Stainless steels offer significantly improved corrosion resistance, mainly through the addition of chromium. Stainless steels typically have 10% chromium content but can contain anywhere from 4% to 30%. The chromium allows the formation of a thin and protective film of chromium (III) oxide no more than 5 nm thick on the surface of the alloy. The film can repair itself provided there is oxygen. In addition to enhanced corrosion resistance properties, they can show superior ductility and toughness when compared to conventional steels.

Because of their untarnished appearance, particularly the ferritic and austenitic grades, they are often employed for aesthetic uses where the additional cost may be easier to justify. The inclusion of significant levels of chromium and other relatively expensive alloying elements increases their value, both as stock material and as scrap for recycling, with the potential negative consequences of theft and pilfering.

In addition to chromium, a range of other alloying elements are commonly encountered in stainless steels, notably nickel but also including molybdenum, titanium and copper as well as carbon and nitrogen as would be found in conventional steels. Three basic types of stainless steel are available, identified by their crystal structure:

Austenitic, which includes the well-known and respected 18-8 grade,
Ferritic, such as the purposely confusing 18-0 grade used in utility cutlery, and
Martensitic, a heat-treatable engineering steel, often with little or no aesthetic appeal, but good for making cutting blades.

In addition to these three main types there are two variants, precipitation hardened and duplex or superduplex stainless steels (Beddoes and Parr, 1999).

Because of their improved corrosion properties, stainless steels have the potential to offer a more sustainable option where long, low-maintenance service life is of benefit. There is an additional cost but this is usually more than offset by the longer life and reduced need for additional protection measures such as coatings. Given their potentially greater importance as we move away from a period of built-in obsolescence and a throwaway culture, it is worth considering this family of materials in greater depth.

6.3.1 Ferritic stainless steel

Ferritic stainless steels are magnetic, non-heat-treatable steels that contain 11–30% chromium but little or no nickel. They are typically employed for nonstructural applications where there is a requirement for good general corrosion resistance or good stress corrosion resistance (an unfortunate synergy of stress and corrosion leading to accelerated failure), such as in seawater applications. They can be particularly useful for internal and decorative applications where aesthetics are the main consideration.

Ferritic stainless steels have good heat and corrosion resistance; in particular they are highly resistant to chloride stress corrosion cracking although the mechanical properties, such as low-temperature toughness, are generally poorer than austenitic grades. The corrosion performance is also poorer than the austenitic grades, and they are more difficult to weld with a greater risk of weld sensitisation unless stabilised by alloy additions. With a maximum nickel content of 0.5%, they are significantly less expensive than austenitic grades and less prone to the price fluctuations associated with the volatile international nickel market. Table 6.1 gives the composition for a range of ferritic stainless steel grades, together with a range of international classifications. Where grades are quoted in the text, the European Standard (EN) classification is given, followed by the American Iron and Steel Institute (AISI) equivalent in brackets.

The most commonly used grades of ferritic stainless steels are Types 1.4512 (409) and 1.4016 (430). Type 1.4512 is titanium stabilised to prevent sensitisation of welds and is, for example, commonly employed in stainless steel car exhaust systems. Type 1.4016 is more commonly employed for cheap cutlery, car trims and other everyday applications where weldability is unlikely to be an issue.

Where enhanced corrosion resistance is required, Type 1.4113 (434) may be more appropriate and is suitable for applications requiring deep drawing such as the manufacture of thin-walled cylindrical items. The so-called low interstitial grades of ferritic stainless steel contain less than 0.03% carbon and nitrogen which enhances ductility, toughness and weldability. Welding conventional ferritic stainless steels can cause excessive grain growth, sensitisation and a decrease in ductility. Alternatively, the use of heat treatment after welding can minimise such problems, even when using the standard ferritic grades (Lula, 1982).

Table 6.1 Classification and composition of common ferritic stainless steels

EN		BS	AISI	Typical composition (%)				
Number	**Name**			**C**	**Cr**	**Ni**	**Mo**	**Other**
1.4000	X6Cr13	403S17	410S	0.08	12			
1.4002	X6CrAl13	405S17	405	0.08	12			0.2 Al
1.4003	X2Cr11			0.03	11	0.5		
1.4016	X6Cr17	430S17	430	0.08	17			
1.4113	X6CrMo17-1	434S17	434	0.08	17		1	
1.4509				0.015	18			Nb, Ti
1.4510	X6CrTi17		430Ti	0.05	17			0.6 Ti
1.4511	X6CrNb17		430Nb	0.05	17			0.6 Nb
1.4512	X6CrTi12	409S19	409	0.03	11			0.5 Ti

6.3.2 Austenitic stainless steel

Austenitic stainless steels have traditionally been the most widely used grade for general applications, although recent increases in the cost of nickel have resulted in it being relatively expensive and very variable in cost. The nickel, up to 10%, is added to ensure the formation of the austenitic structure.

While relatively soft and weak in their annealed condition, austenitic stainless steels can be strengthened by cold working, precipitation hardening or through the addition of nitrogen. They have good ductility and toughness, even the high-strength grades, and their most notable feature is that they are nonmagnetic. One consequence is that they can be used in the construction of buildings where large magnetic fields will be generated, for example, MRI medical scanner facilities.

They retain these properties over a wide range of temperatures, often right down to cryogenic levels. A potentially negative property is that they cannot be strengthened by heat treatment. Austenitic grades can display excellent corrosion resistance but are susceptible to stress corrosion cracking in certain specific environments.

Conventional chromium–nickel austenitic stainless steels include the well-known 18-8 (18% chrome, 8% nickel) grade. The chromium to nickel ratio can be optimised to improve formability and the carbon content reduced to enhance intergranular corrosion resistance. Molybdenum is often added to enhance the corrosion resistance, in particular the ability to resist pitting in environments containing chlorides. It is possible to partly replace the relatively expensive nickel with cheaper manganese or nitrogen additions to produce lower cost and lower performance chromium-manganese-low-nickel grades such as 1.4372 (the old 201 grade). Table 6.2 gives the composition for a range of austenitic stainless steel grades, together with a range of international classifications. Table 6.3 shows how the properties of a standard 18-8 austenitic stainless can be modified by changes in alloy composition.

Type 1.4301, more widely recognised as 304 or 18-8, has good corrosion resistance under atmospheric conditions and is commonly specified for architectural, food and beverage, chemical processing equipment and construction applications. The addition of 2%–3% molybdenum found in Type 1.4401 (the old 316 grade) improves the pitting and crevice corrosion resistance and is used for similar applications to Type 1.4301 but where there are more aggressive conditions that could lead to pitting, such as marine environments.

Austenitic stainless steel is also used to produce reinforcement for use in concrete complying with the requirements of BS 6744:2001 and other international standards. Bars are available in Types 1.4301 (304) and 1.4401 (306), the latter being for particularly aggressive environments such as marine applications. Because of the relatively high and variable cost of austenitic grades, it is possible to obtain clad reinforcement with a stainless outer layer and a carbon steel core, although precautions must be taken to avoid bimetallic corrosion.

Austenitic stainless steel is also used in many brickwork and façade support systems and is widely used for the manufacture of brick ties where they overcome many of the problems of premature corrosion encountered by earlier galvanised brick tie systems.

The familiar but now renamed 200 and 300 series austenitic grades are the most weldable types of stainless steels, especially in their low carbon grades originally

Table 6.2 Classification and composition of common austenitic stainless steels

EN				Typical composition (%)				
Number	**Name**	**BS**	**AISI**	**C**	**Cr**	**Ni**	**Mo**	**Other**
1.4301	X5CrNi18-10	304S31	304	0.07	18	8		
1.4303	X5CrNi18-12	305S19	305	0.06	18	11		
1.4305	X10CrNiS18-9	303S31	303	0.10	18	8		0.35x S
1.4306	X2CrNi19-11		304L	0.03	18	10		
1.4307		304S11	304L	0.03	18	8		
1.4310	X12CrNi17-7	301S21	301	0.05–0.15	17	6		
1.4311	X2CrNiN18-10	304S61	304LN	0.03	18	9		0.22x N
1.4372			201	0.15	17	4.5		6.5 Mn
1.4401	X5CrNiMo17-12-2	316S31	316	0.07	17	11	2	
1.4404	X2CrNiMo17-13-2	316S11	316L	0.03	17	11	2	
1.4406	X2CrNiMoN17-12-2	316S61	316LN	0.03	17	11	2	0.22x N
1.4432		316S13	316L	0.03	17	11	2.5	
1.4435	XCrNiMo18-14-3	316S13	316L	0.03	17	13	2.5	
1.4436	X5CrNiMo17-13-3	316S33	316	0.05	17	11	2.5	
1.4438		317S12	317L	0.03	18	13	3	
1.4439	X2CrNiMoN17-13-5			0.03	17	13	4	0.22x N
1.4541	X6CrNiTi18-10	321S31	321	0.08	18	9		0.5 Ti
1.4550	X6CrNiNb18-10	347S31	347	0.08	18	9		0.5 Nb
1.4567		394S17	304Cu	0.04	18	9		4x Cu
1.4571	X6CrNoMoTi17-12-2	320S31	(316Ti)	0.08	17	11	2	0.5 Ti
1.4539	X1CrNiMoCuN25-20-5	904S13		0.02	19	24	4	2x Cu
1.4547				0.02	20	18	6	1x Cu
1.4529	X1CrNiMoCuN25-20-6			0.02	19	24	6	1.5x Cu

Table 6.3 **Modifying the properties of common austenitic grades**

Requirement	Action	Example grades
Reduce risk of sensitisation	Lower C content	1.4307, 1.4401, 1.4438
	Add Ti	1.4541
	Add Nb	1.4550
Increase strength	Add N or Mo	1.4307
	Precipitation harden	2S143
Increase machinability	Add S	1.4305
	Add Se	Old BS 303 Se (No EN)
Increase oxidation resistance	Vary Cr and Ni content	1.4833, 1.4845
Reduce pitting attack	Add Mo	1.4401, 1.4438
Increase creep resistance	Add Nb	1.4550
Cheapen	Lower Ni, add Mn/N	1.4372

designated by an additional L. However, at the high temperatures involved in welding, sensitisation may occur where some of the chromium reacts with carbon in the steel to produce chromium carbide while depleting the chromium content of the surface. This can lead to severe localised corrosion but may be avoided if the stainless steel has a very low carbon content (<0.03%), such as Types 1.4307 (304L) and 1.4404 (316L), or if the steel is stabilised with titanium, Type 1.4541 (321), or niobium, Type 1.4550 (347).

Aside from the concerns of bimetallic corrosion, differences in the coefficients of thermal expansion between austenitic stainless and carbon steels mean that precautions must be taken to avoid distortion and cracking if these materials are joined by welding (Marshall, 1984).

6.3.3 Martensitic stainless steel

Through quenching and tempering, martensitic stainless steel can achieve greater hardness and hold a sharp edge, a significant advantage over the austenitic and ferritic grades. The downside is that the presence of the carbides that provide these characteristics reduces their corrosion resistance and ability to be easily formed. They have even poorer corrosion resistance than ferritic grades. Their ability to hold a sharp edge makes them ideal for knife blades and scalpels. The combination of good corrosion and wear resistance and high strength means they are suitable for valves, pumps and other similar machinery.

Martensitic stainless steels have between 11% and 18% chromium but can have up to 1% carbon, which imparts the hardness and cutting ability. Occasionally they will also include nickel or molybdenum. The lower chromium grades are generally not suitable for atmospheric exposure as they will tarnish or corrode and are more likely to be used in machine applications where they have additional protection such as oil or grease. Table 6.4 gives the composition for a range of martensitic stainless steel grades, together with a range of international classifications.

Amongst the other characteristics of martensitic stainless steels are that they are magnetic and have good fatigue properties once heat treated. They generally do not

Table 6.4 Classification and composition of common martensitic stainless steels

EN				Composition guide				
Number	**Name**	**BS**	**AISI**	**C**	**Cr**	**Ni**	**Mo**	**Other**
1.4006	X10Cr13	410S21	410	0.08–0.15	12			
1.4005	X12CrS13	416S21	416	0.08–0.15	12			0.35x S
1.4021	X20Cr13	420S29	420	0.16–0.25	12			
1.4028	X30Cr13	420S45	420	0.26–0.35	12			
1.4029		416S37	416	0.25–0.32	12			0.35x S
1.4034	X46Cr13			0.43–0.50	13			
1.4057	X20CrNi17-2	431S29	431	0.12–0.22	15	2		
1.4104	X12CRMoS	416S29	416	0.10–0.17	16		0.4	0.35x S
1.4112	X90CrMoV18		440B	0.85–0.95	17		1.0	0.10 V
1.4116	X45CrMoV15			0.45–0.55	14		0.6	0.10 V
1.4122	X35CrMo17			0.33–0.45	16		1.0	
1.4125	X105CrMo17		440C	0.95–1.20	17		0.6	
1.4418	X4CrNiMo16-5			0.06	15	4	1.0	0.02 N
1.4542	X5CrNiCuNb17-4		630	0.07	17	4		4.00 Cu
1.4568			631	0.09	17	7		1.00 Al
1.4594		460S52		0.70	14	5	1.5	1.50 Cu

have particularly good low-temperature properties and would not normally be used for cryogenic applications as they soon become brittle at low temperatures. Heat treatment, while improving strength and fatigue resistance, also reduces the toughness, and they can also become brittle at temperatures above 430°C. Welding needs to be carried out carefully due to their low ductility, and the high carbon content of many grades makes them unsuitable for welded applications.

6.3.4 Precipitation-hardening stainless steel

The addition of elements such as copper, aluminium, niobium and tantalum to chromium–nickel grades can lead to the formation of precipitates which in turn produce a high-strength stainless steel, in some cases stronger than the martensitic grades. These so-called precipitation-hardening (or PH) stainless steels have better ductility compared to martensitic grades of similar strength. They are also more suitable for welding and can display good corrosion resistance at elevated temperatures.

Where the PH grades really show an advantage is that they can be supplied in the solution-treated condition from which they can be machined prior to a relatively low temperature heat treatment, or ageing, which imparts the increased strength with little or no distortion. They are employed where components need to have high strength, high-temperature resistance and good corrosion resistance and are commonly used for gears, fasteners and aircraft components, as well as the inevitable cutlery applications. If exposed to salty or sulphurous environments they can tarnish and are not typically selected for their aesthetic appearance (Zubek, 2006).

6.3.5 Duplex stainless steel

Duplex stainless steels have structures that combine aspects of ferritic and austenitic or martensitic grades. The resultant alloys are designed to display a favourable combination of the parent grades. For example, the austenitic/ferritic duplex steels have higher strength and better toughness than ferritic grades, are not susceptible to stress corrosion cracking, achieve better pitting resistance and can be welded relatively easily. On the downside, their resistance to crevice corrosion is poorer than the ferritic or austenitic grades with similar levels of chromium and molybdenum. The higher performing high-chromium grades are often designated as superduplex.

Duplex and superduplex stainless steels are widely used in the highly demanding oil, gas, paper and petrochemical industries but have been less well known in many more traditional applications. However, increases and fluctuations in the price of nickel and the availability of so-called lean duplexes has meant that it can be more cost effective to use a lower performing duplex grade than the more traditional and familiar austenitic stainless steel, even for applications such as reinforcing bars for concrete. Grade 1.4462 (2205) is now quite commonly employed as an alternative to molybdenum austenitic grades such as 1.4401 (316). The duplex grade can offer similar if not superior corrosion performance, with less than half the nickel, plus it has the added advantage of being heat treatable (Lula, 1983).

Table 6.5 **Classification and composition of common duplex stainless steels**

EN		BS	AISI	Composition guide				
Number	**Name**			**C**	**Cr**	**Ni**	**Mo**	**Other**
1.4362	X2CrNiN23-4		2327	0.03	22	4	0.4	0.4 Cu
1.4410			2328	0.03	24	6	3	
1.4460	X4CrNiMoN27-5-2		2324	0.05	25	5	1.5	
1.4462	X2CrNiMoN22-5-3		2205	0.03	22	5	3	
1.4501		318S13		0.03	24	6	3	0.5 W
1.4507				0.03	24	6	3	1.0 Cu

Table 6.5 gives the composition for a range of duplex stainless steel grades, together with a range of international classifications.

6.4 Weathering steels

Weathering steels, sometimes referred to as high-strength low-alloy (HSLA) steels, are formulated to produce a dense, stable oxide layer that provides sufficient protection without the need for coating systems. This oxide layer results from the inclusion of alloying elements such as copper, chromium, nickel and phosphorous and is comparable to the patina seen on uncoated cast iron exposed to the atmosphere.

For the protective oxide layer to form, the steel needs to be exposed to cycles of wetting and drying. It is important that ponding of water and exposure to chloride ions are avoided as these can prevent the steel from being adequately protected and lead to unacceptable corrosion rates. A further limitation to the use of weathering steels is their appearance. The patina is visually similar to brown rust and can stain adjacent surfaces which may be problematic where aesthetics are a consideration and makes visual inspection somewhat more difficult. On the other hand, a number of iconic sculptures have been produced from this material, including the ‘Angel of the North’ in Gateshead, UK, and the Picasso sculpture in Chicago, USA (McKenzie, 1978). Being essentially a conventional steel, reuse and recycling is largely unaffected.

6.4.1 Optimising and accelerating patination

The surface patina formed on weathering steel in atmospheric conditions provides protection against further corrosion, a benefit which the corrosion products formed on carbon steel do not provide. The corrosion protection provided by the weathering steel patina is due to the dense and adherent nature of the corrosion products, which act to significantly reduce the rate of further corrosion. The formation of a good-quality stable and corrosion-resistant patina will occur when the steel is exposed to repeated cycles of surface moisture and drying. A protective layer will not form if the surface is continuously wet.

Depending on the conditions, it may take several years for a dense and stable patina to form, which is then able to reduce further corrosion to a low rate. It is therefore desirable to establish a method by which a high-quality patina can be created in a short period of time. Pre-patination of weathering steel components would help achieve a homogeneous appearance and reduce runoff of corrosion product onto surrounding materials resulting in less staining (see Fig. 6.1).

It is possible to apply a chemical treatment to the surface of the steel to encourage rapid development of a corroded surface, resembling the appearance of the naturally aged patina. These are typically combinations of acids and corrosive agents such as chlorides but are not available as warranted systems with technical support. As a consequence, such chemical treatments are typically only employed on sculptural pieces and are unlikely to result in the development of the long-term protective properties required in structural applications.

A patination treatment for weathering steel has been available for a number of years in Japan. It works in much the same way as patination oil for lead in that it allows the stable oxide layer to form beneath a protective coating that impedes less desirable forms of surface corrosion. Unlike patination oil, the short-term effect is not visually pleasing and results in the elements appearing to have been whitewashed. The coating slowly chalks away over a period of years until finally a perfectly formed patinated surface is exposed (Kage et al., 2005).

The alternative to chemical or coating systems is to employ carefully controlled cycles of wetting and drying to allow the rapid but natural development of a patina. Such a service is available commercially in the USA, although no similar facility is known to be available in the UK or Europe, and the approach only works effectively when fully fabricated components, such as cladding panels, can be treated.

Fig. 6.1 Patina forming on weathering steel.

6.5 Nonferrous metals and alloys

While the vast majority of metal use is centred on ferrous-based alloys, there are significant amounts of other materials employed for specific applications where weight, strength or chemical resistance are important.

6.5.1 Aluminium

Aluminium is the most abundant metal in the Earth's crust. Despite this, it was only comparatively recently that it could be produced and employed to any significant extent. It is a highly reactive element and is found in a wide range of naturally occurring compounds. The most common aluminium ore is bauxite but obtaining the metal from it is far from a straightforward matter.

The metal was first identified in the early 19th century and obtained the name *aluminum* from Sir Humphry Davy. This was later modified to aluminium when elemental names were unified, although in the USA it was decided as recently as 1925 to revert to the original spelling. Commercial production of aluminium only became possible through the development of an electrolytic process which started with Davy but became commercially viable in the late 1880s through the Hall-Héroult process.

The method of production of aluminium has always been relatively energy intensive. The production of iron from ore uses approximately a quarter of the energy required to manufacture aluminium from raw products. When recycled from existing metal, both aluminium and ferrous materials require the same energy input by mass. As a consequence, aluminium with its lower bulk density can be seen as a more sustainable recycling option than steel.

Aluminium and its alloys generally display good corrosion resistance which further enhances their sustainability credentials. A large proportion of aluminium applications can take advantage of aluminium's ability to protect itself from most environments through the formation of a protective oxide layer. This is artificially thickened through the process known as anodising. Unlike weathering steel, the protective oxide layer is essentially invisible and can be modified with dyes and surface treatments to achieve a range of finishes.

Despite these apparent advantages of aluminium, steel remains dominant in most areas of construction and engineering due to its superior mechanical properties and better fire resistance, the melting point of aluminium being 660°C compared with around 1370°C for steel (Polmear, 1995).

6.5.2 Copper and copper alloys

The first discovery of copper may well have been over 10,000 years ago in the Middle East. The technology for its manufacture and fabrication were well advanced by the time of the Pharaohs, with 5000-year-old copper pipes still to be found in Egyptian tombs. Copper could be produced by a development of the smelting process first used for iron, but the metal itself was soon recognised for its superior malleability, durability and appearance.

The principal alloys of copper, brass (copper and zinc) and bronze (copper and tin) appear to have been developed in parallel. The Egyptians were the first to recognise that bronze was better for casting than copper and was harder and more durable. Today the term *bronze* is employed to describe a wide range of copper-based alloys such as aluminium bronze, silicon bronze and manganese bronze, each of which have special engineering characteristics.

Copper and its compounds are essentially toxic and can result in similar symptoms to arsenic poisoning. The levels of copper in drinking water are carefully monitored, although under normal circumstances copper is not particularly soluble. This toxicity makes copper valuable for antifouling applications (copper bottoms on timber ships) and antibacterial applications such as copper doorknobs in hospitals which can limit the spread of disease. Where copper sheeting is used as a roofing material, consideration must be given to the possibly toxic qualities of the runoff water.

Conductivity, both electrical and thermal, is the other characteristic that makes copper so useful and valuable. It is second only to silver in terms of electrical conductivity and is a vital component in electrical power and signal transmission systems. Recycling of copper conductors is big business and the illicit stripping of copper wiring of increasing concern, particularly as it often involves the removal of polymer sheathing by burning, resulting in the release of potentially toxic fumes.

Atmospheric corrosion of copper alloys produces a characteristic blue/green discolouration. When used for aesthetic purposes, copper and its alloys usually require to be either protected with a transparent lacquer or have the surface modified with a chemical patination treatment that is designed to produce a stable and attractive finish. Without such treatments it can take many years for copper roofs and bronze statuary to take on the characteristic appearance of antiquity (Copper Development Association, 2004).

6.5.3 Lead

As with copper, the use of lead dates back thousands of years. Its early popularity was based on its relative ease of manufacture from widely available lead ores, low melting point, ease of fabrication and good corrosion resistance. Lead pipes were used for drinking water by the Romans and is the source of the common terms *plumbing* and *plumber*. Lead oxides were widely used as pigments in drying oil paints, where they assisted in the through-hardening of the paint film. One further characteristic of lead that is difficult to replicate with other materials is its ability to act as a shield to radiation, particularly in hospital and nuclear applications.

Health concerns with respect to the effects of cumulative lead poisoning have resulted in the cessation of the plumbing and painting options, although many examples still exist in old buildings with the associated risks. For centuries lead sheet was the ultimate roofing system, although traditionally prey to unofficial and illegal recycling, especially from church roofs.

Lead continues to have a role in construction where its durability, formability and appearance are considered of value, such as with flashing. Some roofing is still carried out, particularly on historic structures. Lead-coated steel, or terne plate, is used for decorative roofing and cladding, although fears over lead exposure have resulted in much of it being replaced by zinc-coated alternatives.

When lead is exposed to the air it produces a characteristic layer of oxides and carbonates that gives the familiar lead patina finish. If left to occur naturally, this can be uneven in appearance and result in staining of adjacent masonry; it is therefore preferable to treat new lead with patination oil, a mixture of linseed oil and other compounds, which encourages the formation of a stable, even and aesthetically pleasing surface finish (Blaskett and Boxall, 1990).

6.6 Corrosion

As previously discussed, steel has a similar life cycle to the people who use it. The length of time between ‘conception’ and ‘death’ is influenced by how well they are treated and how appropriately they are employed. For steel and other oxidisable metals, corrosion represents the biggest threat to a long and healthy life. For our purposes we will mostly concentrate on steel, being the dominant material used in construction, and the effects of aqueous corrosion where moisture and oxygen are involved (Lambert, 2005).

At the most basic level the corrosion process can be demonstrated by two dissimilar metals in an aqueous electrolyte that have been electrically connected to allow a flow of electrons from one to the other. One metal, the more reactive one, becomes the anode and essentially dissolves. The other, the cathode, remains unaffected but is essential for the corrosion process to continue. The reactions occurring at these anodic and cathodic sites are given here.

At the anode

$$\mathrm{M} \rightarrow \mathrm{M}^{n+} + n^{\mathrm{e}-}$$

$$\{\text{Metal} \rightarrow \text{metal ion} + \text{electrons}\}$$

At the cathode

In well-aerated neutral and alkaline environments,

$$\mathrm{O}_2 + 2\mathrm{H}_2\mathrm{O} + 4^{\mathrm{e}-} \rightarrow 4\mathrm{OH}^-$$

$$\{\text{Oxygen} + \text{water} + \text{electrons} \rightarrow \text{hydroxyl ion}\}$$

In some cases, especially in acidic conditions or in the absence of oxygen, the following reactions can occur:

$$2\mathrm{H}_3\mathrm{O}^+ + 2^{\mathrm{e}-} \rightarrow \mathrm{H}_2\mathrm{O} + \mathrm{H}_2\ (\text{gas})\quad (\text{acidic conditions})$$

$$2\mathrm{H}_2\mathrm{O} + 2^{\mathrm{e}-} \rightarrow 2\mathrm{OH}^- + \mathrm{H}_2\ (\text{gas})\quad (\text{neutral conditions})$$

The dissolved metal ions react with hydroxyl ions to form corrosion products such as rust, as shown in Fig. 6.2. Production of hydrogen at the cathode can lead to hydrogen embrittlement and possible failure in some sensitive materials, for example, HSLA steels that are under stress.

It is not necessary for there to be two different metals joined together, as anodic and cathodic areas can be formed on a single surface in contact with the aggressive aqueous environment. Small differences in grain orientation, composition, hardness, moisture, oxygen or temperature are all that is required to differentiate cathodic and anodic areas. Corrosion can therefore occur at a large number of sites over the surface of the metal, each with their own local anodes and cathodes, as illustrated in Fig. 6.3.

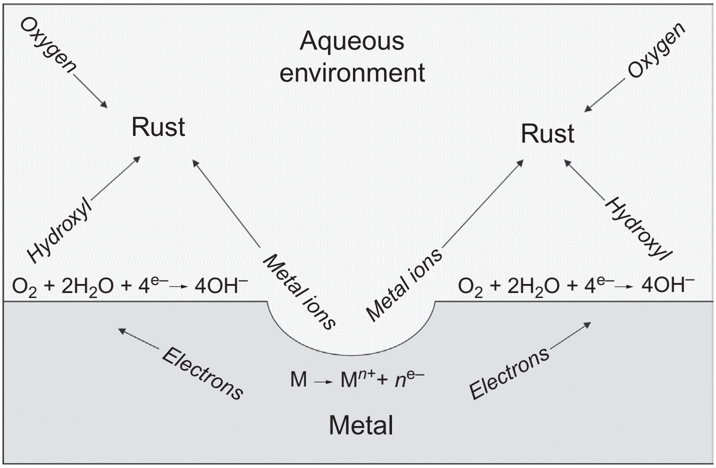

Fig. 6.2 A simple corrosion cell.

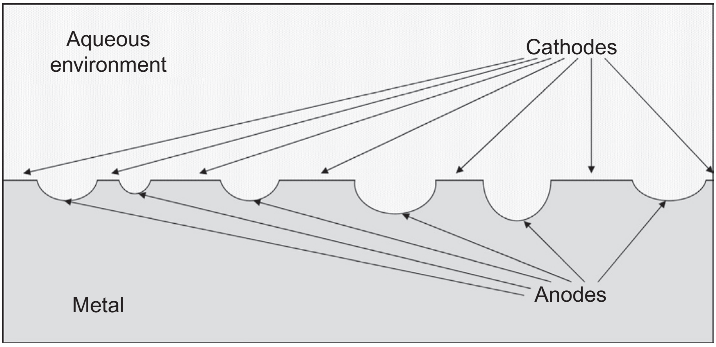

Fig. 6.3 Anodes and cathodes.

6.6.1 General corrosion

General corrosion results in what at least appears to be a uniform attack on the metal surface. Closer examination will often identify an ‘orange peel’ effect, the dimples being individual anodes surrounded by cathodic areas. It is worth noting that the corrosion product or rust is deposited over the whole of the affected area, not just where the anodes are located.

Because the corrosion is evenly spread, the overall rate of penetration is generally quite low but can generate a large amount of corrosion product, which may cause problems with aesthetics and contamination. It is possible to obtain typical corrosion rates for most commonly employed alloys in specific environments.

6.6.2 Pitting corrosion

Under particular circumstances corrosion can be localised in a small number of sites to produce pits. While the total amount of corrosion may be similar to that encountered with the general form, because it is more focussed the corrosion rate is much higher and can lead to significant loss of section or perforation. Pitting corrosion can occur due to a number of causes but one of the most commonly recognised is due to chlorides, where the chloride ions locally disrupt the passive oxide film that normally protects metals. Pitting can also occur when there is a susceptible phase in a metal which is preferentially attacked.

Once initiated, a chloride-induced pit can be self-perpetuating, with chloride never actually being consumed but rather acting as a catalyst. Pitting is often associated with stagnant conditions, which can cause problems when the plant is idle or closed down for maintenance.

6.6.3 Crevice corrosion

Crevices are very effective initiators of corrosion. Wherever there is a fine gap, such as between two flanges, there is the risk of crevice corrosion being initiated. It is not necessary for both surfaces to be metals, and crevices can even form beneath surface deposits. The crevice retains water which quickly attains a lower oxygen content than outside the crevice. Even if the item in question is not submerged, the crevice will retain water through capillary attraction. The difference in oxygen level between the crevice and exterior causes a corrosion cell to be formed, the part in the crevice being the anode as illustrated in Fig. 6.4.

The relatively small anodic area is driven by the larger external cathode, resulting in high rates of corrosion. Unfortunately, the smaller the crevice, the more intensive the likely corrosion so the only way to deal with crevices effectively is to remove them completely, for example by replacing bolted connections with welded ones.

6.6.4 Galvanic corrosion

The formal recognition of galvanic or bimetallic corrosion dates back to 1763 when the iron nails securing the copper bottom of *HMS Alarm* were observed to have preferentially corroded. That said, all early attempts at producing electricity electrochemically

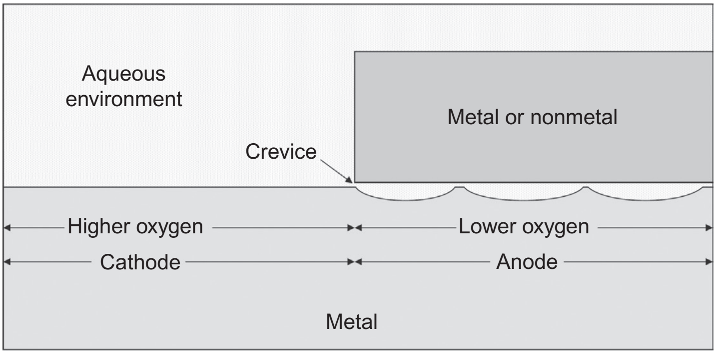

Fig. 6.4 Crevice corrosion.

with batteries employed this principle. It occurs when two dissimilar metals exposed to the same electrolyte are connected electrically, as previously discussed when describing a simple corrosion cell. Batteries and corrosion are essentially the same technology, although the polarity of the anodes and cathodes has been switched as when the function of batteries was first being described, for example by Benjamin Franklin; it was believed that electricity flowed through the movement of positively charged particles rather than electrons.

Galvanic corrosion will only occur when the following conditions are satisfied:

(1) The anodic and cathodic metals must be in the same electrolyte, which need not be particularly aggressive to either of them. The electrolyte can be a solution, condensation film or damp solid (including soil or concrete), the only requirements being that it contains moisture and dissolved ions.

(2) The anodic and cathodic reactions on the two metals must be able to proceed. In particular, the anodic metal must be susceptible to corrosion in the environment.

(3) The two metals must be connected electrically, either directly or indirectly via a third metal or, for example, earthing strap. Indeed, problems occur when components of different metals that have been purposely isolated to prevent galvanic corrosion are subsequently both bonded to earth for electrical safety.

(4) The two metals must have a sufficiently large potential difference to drive a significant galvanic current. This can be evaluated by comparing their position in an electrochemical series, although it must be noted that these are specific to the environment (Table 6.6 shows one for seawater; the values are in volts measured against a standard hydrogen electrode) and metals such as aluminium appear worse than might be expected as the effects of their protective oxide film resulting from anodising are not taken into account.

6.6.5 High-temperature corrosion

In addition to the more familiar ‘wet’ corrosion, metals can lose section through scaling in dry, high-temperature environments. At elevated temperatures, steel and other metals can react with oxygen or gaseous oxides of sulphur and carbon, resulting in loss of metal, generation of corrosion products and possible changes in the mechanical characteristics of the alloy.

Table 6.6 **Electrochemical series for seawater**

Metal	Potential, *E* (V)
Gold	+1.50
Silver	+0.80
Mercury	+0.79
Copper	+0.34
(Hydrogen)	0
Lead	−0.13
Tin	−0.14
Nickel	−0.25
Cadmium	−0.40
Iron	−0.44
Chromium	−0.74
Zinc	−0.76
Titanium	−1.63
Aluminium	−1.66
Magnesium	−2.37

High-temperature oxidation and other associated reactions have historically been problems for the power generation, aerospace and petrochemical industries. Alloys containing high chrome and especially high nickel contents can deliver superior performance at elevated temperatures but are inevitably expensive. In recent years more cost-effective solutions have been available through the use of engineering ceramics.

6.7 Corrosion protection and prevention

Despite the propensity for metals to corrode, in particular the ferrous alloys, the benefits of using such materials make it worthwhile to control the corrosion processes by means of various methods that interfere with the formation of corrosion cells. These can range from simple barriers that exclude one or more of the corrodents, to electrochemical systems that influence the formation of anodes and cathodes.

6.7.1 Protective coatings

The most commonly employed method of protecting metals from corrosion is to apply a protective coating. By applying a thin adherent coating to the metal, moisture and aggressive species such as chloride ions are kept away from the surface, and corrosion does not occur. In reality, most protective coating systems are more complex than simple barriers. They contain active species that enhance the bond to the metal surface and special fillers to increase the barrier properties. Coatings also provide a wide range of aesthetic options, particularly for ferrous materials that would otherwise come in any colour as long as it was a rusty brown.

Coating systems have changed dramatically in the last few years as many of the active ingredients were also harmful to the environment, the applicators or the public. Anti-corrosive components based on lead and chromium have been largely phased out and highly solvented systems are being replaced by water-based coatings which benefit the health of both the applicators and the environment.

Various attempts have been made to use coatings to impart additional protection to reinforcing bars for concrete. The two most popular systems have been zinc galvanising and fusion-bonded epoxy. While each continues to be used in niche applications and more widely in specific parts of the world, they also can have considerable drawbacks. These are notably the reactivity of zinc in alkaline environments and the sensitivity of epoxy to construction damage and tendency to soften with time.

6.7.2 Design and material selection

Design plays an important role with respect to controlling corrosion. It is essential to avoid the trapping of dirt and water and ensure drain holes will not get blocked. As previously discussed, crevices should be avoided at all costs.

A number of common approaches to preventing galvanic corrosion may be employed; these include insulating the metals from each other, applying coatings to impede the anodic and cathodic reactions or by applying cathodic protection as described later. Jointing compounds that exclude water and do not dry or crack can be used to protect joints. If the compounds have a corrosion inhibitor incorporated into them, they may be effective in moderate conditions. If possible the compound should be overcoated.

When considering stainless steels, some grades are clearly more desirable to use than others for certain applications. In the construction industry the austenitic group of stainless steels is used most widely in applications such as dowels, fixtures, bolts, fasteners and reinforcements. Care must be taken in the choice of stainless steels to be used in specific situations as many problems such as galvanic corrosion, stress corrosion cracking, crevice corrosion, intergranular corrosion, pitting and galling may be encountered if the chosen grade is inappropriate (Revie, 2008).

6.7.3 Cathodic protection

On the basis that corrosion relies on two processes occurring simultaneously, the anodic and cathodic reactions, and that only the anodic reactions result in a loss of metal, it is possible to prevent corrosion by making all surfaces of the susceptible metal act as a cathode. The term *cathodic protection* was first coined by the ubiquitous Humphry Davy in 1824 when he described his concept for preventing the preferential corrosion of iron nails used to fix the copper sheeting to the bottoms of timber ships. As has already been described, the resulting galvanic corrosion caused the heads of the nails to fall off and the copper sheeting to become detached.

Davy came up with the idea of fixing lumps of a third metal more likely to corrode than either the iron nails or copper sheet that would ‘sacrifice’ itself and prevent the nails from rotting away. Unfortunately, the original system was too effective and also prevented the corrosion of the copper, which was required for anti-fouling properties. It was to be another hundred years before the real benefits of cathodic protection were identified, and it is now unusual for a metal pipeline, ship, jetty or oil rig to not be protected by such a system.

The technique relies on the metal to be protected being in an electrolyte and cannot therefore be used to protect atmospherically exposed components and structures. The electrolyte may be seawater, freshwater, soil, sand, mud, concrete or masonry. The only requirement is that it contains moisture and can carry an ionic charge. While galvanic systems based on the preferential corrosion of a reactive metal (zinc, aluminium or magnesium) are still common, many systems now provide the protection by means of an impressed current.

The characteristic of an anode that is important in this process is the generation of electrons (see Section 6.6). This can be achieved with an inert anode and a DC power supply. The most effective anode material for passing large quantities of current without dissolving is platinum. Thankfully, similar performance can be achieved with relatively inexpensive materials such as mixed metal oxide (MMO)-coated titanium and titania (titanium oxide). Fig. 6.5 shows a simple schematic to illustrate the difference between impressed and galvanic cathodic protection systems.

The construction industry has already made significant progress and associated cost benefits through the use of cathodic protection, mainly for the remediation of chloride-contaminated, reinforced, concrete structures (Chess and Broomfield, 2014). A comparison was carried out for a dual two-lane highway overbridge requiring repairs to two central supports. The cost for conventional repairs, including access and traffic management, was four times greater than for the cathodic protection (CP) option. Elsewhere, the cost of CP was found to be less than 5% the cost of replacing the damaged supports to an elevated section of highway. As well as saving money, the CP

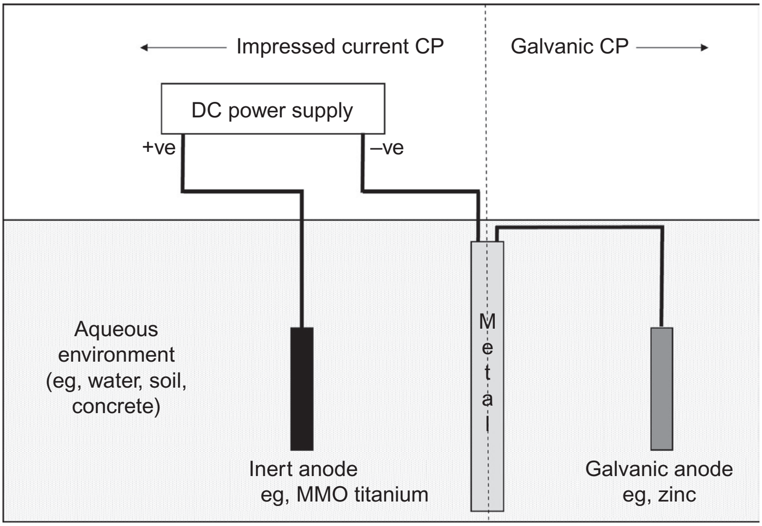

Fig. 6.5 Impressed current and galvanic cathodic protection.

option was found to greatly reduce the carbon footprint for the works and improve health and safety for the workforce, motorists and local residents. Cathodic protection is still only rarely used on new reinforced concrete structures to prevent corrosion from occurring in the first place, although growth in this area has been reported in the hot, salt-laden environment of the Middle East (Arya and Vassie, 2005).

A more recent application for cathodic protection in the construction industry has been in the preservation of historic steel-framed buildings. Many of the grand municipal and commercial buildings constructed in the major cities of Europe and North America in the early part of the 20th century employed a structural steel frame, clad in masonry. This form of construction allowed for taller buildings with larger openings for windows and wide-open floor spaces, better suited for banking and commerce. Cathodic protection solutions for use in such structures have been subject to considerable development and widening use (Corrosion Prevention Association, 2011).

6.7.4 Corrosion inhibitors

A method that is widely used in the protection of metals in domestic, commercial and industrial applications and has found its way into construction applications is the use of corrosion inhibitors. A corrosion inhibitor is essentially any material that when present in a relatively small quantity results in a significant reduction in the corrosion rate of metals, predominantly the ferrous-based materials such as iron and steel. There has long been interest in their potential use for the protection of metallic reinforcement in concrete and other structural materials.

Inorganic inhibitors such as nitrites and benzoates have been used as anti-corrosive admixtures in concrete since the 1950s, principally as admixtures introduced at the time of manufacture. A more recent development has been the development of organic inhibitor systems that can be applied to the surface which then migrate to and protect buried metallic components. These materials are commonly based on amino alcohols and are capable of migrating through masonry and concrete to form a highly adherent microscopic film covering the surface of the metal and thereby protecting it.

Because the surface of the porous material to which they are applied is unaltered, they allow the protection of embedded metals without altering the appearance of the structure. The service life of the inhibitor, typically several years, is dependent on the porosity of the material and severity of the environment but can be easily reapplied. Other recent applications for corrosion inhibitors have included cramps and wall ties and the protection of historically important industrial sites with wax-based corrosion-inhibiting coatings (Lambert and Foster, 2009).

6.8 Future trends

Metals continue to be in high demand, and as more parts of the world aspire to Western levels of personal, corporate and municipal wealth the need for metals is anticipated to increase. Steel supply is particularly stretched given the huge tonnages required in the development of rapidly developing economies such as China and India.

Developments in the use of carbon nanotubes and similar nanotechnologies promise to revolutionise aspects of the electronics industry; however, the amounts of copper and other metals that such developments may replace are likely to be miniscule when compared with the overall demand, plus there remain health concerns regarding risk of inhalation or ingestion that have yet to be resolved. Nano-modified alloys are also claimed to be capable of giving tenfold increases in strength and superior corrosion resistance to conventional materials (Bullis, 2015).

Recycling of metals remains a major industry. Prevention of corrosion, particularly of ferrous materials, needs to be given higher status alongside recycling as a sustainable way of obtaining the maximum benefit from the energy invested in a metallic component. The benefits can already be seen in the motor industry where better design and more effective corrosion protection have made the corrosion of bodywork a relatively minor issue throughout the life of vehicles manufactured within the last 15–20 years.

In the construction industry, the inherent value of recovered structural steelwork and reinforcing bars ensures that they enter the recycling system. Reuse rather than recycling of rolled steel beams and stanchions offers enormous potential savings but remains relatively rare in the absence of guidelines and government incentives.

As the benefits of longer component life become more widely recognised, the use of higher performance materials for sustainable rather than aesthetic reasons should become more common. Materials such as stainless steels and aluminium alloys could have much wider use in construction applications than is currently the case, and in turn provide longer service lives, lower maintenance and ultimately more sustainable solutions.

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Sustainability of timber and wood in construction

7

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7.1 Introduction

Wood is used in almost every country throughout the world, and it has always had a particularly strong social and economic value. Today from a building construction material perspective it is wood's environmental significance, as a sustainable, renewable and natural resource, which is now starting to be fully appreciated and harnessed. Wood buildings and appearance products span the ages: ancient, modern, futuristic. Wood is one of the earliest construction materials and, if appropriately durable species are specified and maintained correctly, can perform adequately for extremely long periods. The well of Altscherbitz, near Leipzig, is thought to be the oldest timber construction still in existence, built over 7000 years ago (around 5600–4900 BC). Another ancient structure is the Horyuji Temple in Nara, Japan, shown in Fig. 7.1. It is a five-storey, 32.45-m, timber pagoda. Dendrochronological analysis shows that the timbers used were felled over 1400 years ago (approximately 594 AD). In terms of modern and future use, there is a current renaissance in tall-wood construction being driven on two fronts: the advent of a range of new engineered wood products (EWPs) and timber construction systems, and an increasing interest by architects and designers in tackling global CO_2 emissions through carbon-positive environmentally green building design.

Carbon dioxide (CO_2), a slightly toxic, odourless and colourless gas, makes up only 0.033% of our atmosphere, yet it alone has revolutionised global environmental strategies and regulation, including environmentally sustainable building policies and practices. Although CO_2 occurs naturally in the atmosphere, it is also produced from the burning of fossil fuels for energy, enhancing greenhouse effects and causing climate change. It is also the life-breath of plants and forests that 'inhale' CO_2, 'exhale' oxygen and store carbon in their woody tissue. Maintaining and preserving forests is not only critical for sustaining human and animal life on this earth, but active sustainable forest management and regeneration also provides humanity with literally hundreds of valuable wood fibre-based products from fuelwood to activated carbon; pulpwood to high-quality paper; nanocrystalline cellulose to ballistic glass; and of course sawlogs to a vast array of sawn, engineered and appearance grade building products.

As the world now moves into a carbon-constrained future, there is a greater appreciation of wood because of its beauty and versatility as a product and its unique environmental credentials. It is a product obtained from a natural resource that grows on solar energy, converting CO_2 to carbon and emitting oxygen; it is renewable, recyclable and biodegradable; its production and processing are energy efficient and

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Fig. 7.1 Horyuji Temple, Nara, Japan.

its use in construction is 'greenhouse positive'. Wood from sustainably managed forests is arguably one of the most environmentally friendly construction materials available.

The biggest issues for wood are not around the products themselves, but access to the wood fibre, addressing forestry-based concerns and managing broadacre resource issues particularly maintenance of biodiversity, land clearing for food production and illegal harvesting of wood products for cash profits. Sustainable forest management is today a key focus for the international wood sector and markets, and to address these issues a major push is being mounted globally around increasing forest and chain of custody certification to provide confidence to designers and consumers that timbers they might use are coming from legal and sustainable sources.

The topic of sustainability is the most significant new issue on the global building agenda, its prominence enhanced by the intense worldwide media and government interest in CO_2 emission reduction. How buildings are constructed and used is extremely important, as buildings have a significant impact on the environment. The United Nations Environment Programme (UNEP, 2009) reports that the building sector contributes up to 30% of global greenhouse gases and consumes up to 40% of all energy. It also consumes around 30% of the world's resources and 12% of its water, whilst producing around 30–40% of the wastes going to landfill.

The building and construction sector was an early mover in embracing environmental change in many parts of the world in response to the objective of greenhouse gas (CO_2) emission reduction. The interest and focus on sustainable design, improved energy efficiency, environmentally enhanced green buildings and green products has grown at a massive rate particularly over the last 15 years. Building-related sustainability activities have included both:

- voluntary market approaches, whose aims have been to promote greener buildings and material choice and to push the boundaries and recognise exemplary practice, not just 'business as usual' (the first Green Building Council was formed in the US in 1993), and
- government regulations, initially focused on end-of-life issues such as recycling and waste reduction strategies, and on reducing the energy impacts of buildings during their operational phase.

Today, in terms of the environmental performance of buildings, the accepted direction in most countries is a focus on a full life-cycle assessment (LCA) approach which looks at all phases of the building's life:

- the embodied impact of the manufacturing of building materials (particularly CO_2 emission impacts);
- the impacts of construction;
- the true impact of the operational phase of the building including CO_2 impacting factors such as heating, cooling, hot water supply, fixed appliances and lighting; and finally
- the impact at the end of the building's life: demolition/deconstruction, reuse of products, recycling of materials and disposal of waste.

The wood and timber products sector strongly supports a LCA approach, because the facts are

- timber products come from trees, a renewable resource, that absorb CO_2 and through the process of photosynthesis convert this to oxygen which is released to the atmosphere, and carbon which is stored in the woody mass of the tree;
- timber products use comparatively little energy in production and so have low levels of embodied CO_2 in manufacture;
- timber wall, floor and roof framing can provide a lightweight approach to construction using minimal amounts of materials compared with the mass material approach such as concrete, although insulation is required in colder climates;
- when timber products are used in a building, the carbon is effectively stored for the life of the product (a typical US home stores around 9 tonnes of carbon);
- at the end of the home's life, timber products have a great opportunity for direct reuse and recycling (doors, windows, floors, etc.) and reprocessing into other wood-based products (see Chapter 8);
- wood products that can no longer be used for any other purpose can be utilised as fuel in biomass energy plants, reducing the reliance on nonrenewable fossil fuels such as coal; the CO_2 released from the burning of the timber goes back to the atmosphere where ultimately it is rapidly absorbed by other growing trees.

Whilst achieving durability and structural reliability levels are taken for granted, the global focus today for designers, specifiers, regulators (and even consumers) is around sustainable low environmental impacting materials and buildings particularly in regard to greenhouse gas emission reductions. Wood has a great story to tell, particularly with the new and exciting range of EWPs now available and which, when understood and appreciated, truly demonstrates why wood is one of the most desired, useful and environmentally friendly construction materials available.

7.2 Forest resources

A discussion about wood and timber products must begin with a discussion about forests. The Food and Agriculture Organisation of the United Nations (FAO, 2010) ‘Global Forest Resources Assessment 2010’ (FAO, 2010) is a 378-page document that deals with issues that go well beyond the scope of this chapter, but it does place the use of forests and wood and timber in construction in a broad global context and deals with issues that are often misunderstood by the general public. The first paragraph of the Foreword provides an excellent global perspective of the triple bottom line pillars of sustainability: social, economic and environmental values.

> *Interest in the world’s forests has grown to unprecedented heights, especially with growing awareness of their role in the global carbon cycle. The possibility of mitigating climate change by reducing carbon emissions caused by deforestation and forest degradation, and by increasing carbon uptake through afforestation and sustainable forest management, highlights the essential role of forests in supporting life on Earth. But forests are more than just carbon. In 2010 we celebrate the International Year of Biodiversity, and we are reminded that forests represent some of the most diverse ecosystems on Earth. In a time of economic crisis, we are also reminded that forests provide employment and livelihoods for a large proportion of the population – especially in developing countries – and often act as an economic safety net in times of need.*

7.2.1 World resources and how they are defined

FAO (2010), Executive Summary, pp. xiii, reports that forests cover 31% of the world’s land area of just over 4 billion hectares. Five countries—the Russian Federation, Brazil, Canada, USA and China—account for almost half the total forest area, 54 have forest on less than 10% of their total land area and 10 countries have no forest at all. The report continues by stating that on a global scale there is net deforestation, principally the conversion of tropical forest to agricultural land, but its rate is decreasing as afforestation and natural expansion of forests continues in some countries. In interpreting the data the terminology needs to be understood.

1. *Deforestation* means a loss of forest because of clearance by people for conversion to other uses, usually agricultural. It also includes natural disasters where the forest is unable to regenerate.
2. *Afforestation* means planting trees on land not previously forested or by natural regeneration as occurs with abandoned agricultural land. It goes on to define reforestation as cutting down forest then replanting or natural regeneration resulting in no change in forest area.

In the decade from 2000 to 2010 global deforestation occurred at a rate of 13 million hectares (ha) compared to 16 million ha in the decade 1990–2000. These losses are offset by large-scale planting whereby net forest losses declined from 8 million ha to 5.2 million ha on the same decade-to-decade basis. According to FAO (2010), Asian and South American forest loss is driven primarily by clearance for agricultural reasons and by illegal logging. Rainforest timbers attract high prices in the wealthier countries, which is very attractive to illegal loggers in the less wealthy nations. Another driving force in the tropical Asian countries involves the clearing of tropical forest for palm oil plantations, which provides immediate profit for the timber logs, then follow-up profits from palm oil sales. Other regions in African and South American countries also have significant deforestation issues, and Oceania, dominated by Australia, has suffered some levels of deforestation because of severe drought and bushfires. From 2000 to 2010, notably in Europe and Asia, forest areas have increased and there has been no or little net loss in North and Central America. There has also been significant afforestation in recent years in China, albeit off a low base. Much of the drive for afforestation, reforestation and sustainable management has come from the demand by consumers for wood-based products.

Forests today are managed for a multitude of uses and values. FAO (2010) (Executive Summary, pp. xxvi) reports that 20% of the world's forests are managed specifically for conservation of biodiversity, soil protection and water, 30% for production, 24% for multiple uses, an additional 4% for social services; and the remaining 23% are either 'other or unknown'. Wood products are included in the 'production' and 'multiple use' categories. At the global level wood removal amounts to around 3.4 billion m^3 annually, almost half of this being wood fuel. However, 'informal and illegally' removed wood is not counted, and therefore total global removal is undoubtedly higher than the figure cited.

Illegal logging is a major global issue and has been a key driver for the introduction of forest and chain of custody certification particularly in the solid wood products sector. Certification provides major assistance in addressing the issues of sustainable and legal timbers.

Forest certification addresses the 'sustainability' question

Forest certification is a voluntary process by which planning, procedures, systems and performance of on-the-ground forestry operations are audited by a qualified and independent third party against a predetermined standard. Forest operations found to be in conformance with the given standard are issued a certificate (hence certification).

Chain of custody certification assists with the issue of 'legality'

Chain of custody certification provides a specific system to track wood products from a certified forest through the processing and marketing channels to the final user; this system can also be audited and certified by a third party.

Around 11% of the global forest areas are certified; however, this percentage is much higher in specific countries focussed on wood production. There are a variety of forest management certification schemes in operation throughout the world. The two most accepted and predominant international schemes are

- the PEFC, Programme for Endorsement of Forest Certification, the world's largest forest certification system with 36 nationally endorsed schemes and more than 260 million ha of certified forests; approximately 60% of the certified forest (PEFC, 2015), and
- the FSC, Forest Stewardship Council, with 182 million hectares of certified forests in 81 countries globally (FSC, 2015)

Certification is much more than a self-justified marketing claim. Rigorous, independent assessment by third-party auditors must be carried out before forest managers can claim that they are certified, and these assessments are subject to scrutiny. Most of the major credible green building rating schemes have a position on timber certification; in most of the world these recognise both the PEFC and FSC schemes. A number of major government reviews have also been undertaken on certification schemes including CPET (Central Point of Expertise on Timber Procurement) by the UK government in 2006, and TPAC (Timber Procurement Assessment Committee) by the Dutch government in 2008; these reviews have also endorsed both the PEFC and FSC schemes. Recognising that certification takes considerable time and effort to achieve, both schemes take the pragmatic approach, in terms of chain of custody certification, that at least 70% of log volume needs to be sourced from certified forests or recycled materials whilst the balance can be sourced from uncertified forest areas as long as it can be demonstrated that they are legal and noncontroversial sources.

7.2.2 Wood and timber products as building materials

The term *wood* used in the chapter title means the tree skeleton (trunk and branches), and *timber* refers to what becomes of it once it is used for building purposes whether in a round, sawn or some other form. The widespread use of timber (often called lumber in Canada and the US) in building construction comes about for the following reasons.

- Globally, forests are relatively plentiful, and wood for timber products can be easily extracted and processed using low levels of technology.
- Provided the wood is sourced from sustainably managed forests, the supply of raw material to the timber industry is assured. In sustainably managed forests the rate of extraction of wood is balanced by replanting and natural regrowth.
- Because timber is lightweight and strong in bending, compression and tension, it is capable of performing all the basic structural functions.
- Enhanced structural performance can also be achieved from the parent wood material when processed into EWPs.

7.3 Timber use and engineering issues

7.3.1 Forms of timber use

Timber is used in a variety of forms:

- Round timber, where the trunk of the tree is used directly with little or no further processing. Typical of this type of use is as stringers in bridge building, as compression elements in housing construction, piles and electric power poles.
- Sawn timber, the predominant form for residential and light commercial framing (walls, floors and roofs) and appearance grade products (flooring, lining, cladding [siding], windows, doors, cabinetry, joinery, etc.), temporary structures to support concrete formwork, and for furniture.
- EWPs, used in both residential and nonresidential construction (apartments, public buildings, offices, sporting stadiums, etc.), which usually involves mechanical fastening of sawn components with metal nail plates (such as nail-plated beams or trusses), or adhesive bonding of flaked, chipped, shaved veneer or sawn timber to form larger timber panel, plate or beams elements. EWPs include: medium-density fibreboard, hardboard, oriented strand board (OSB), particleboard, plywood, trusses, I-beams, laminated veneer lumber (LVL),

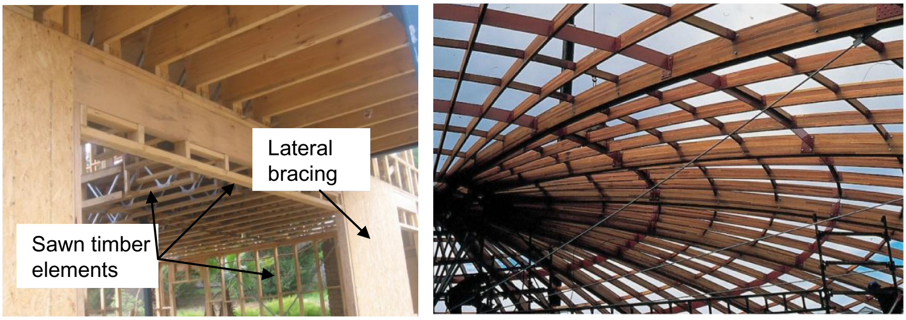

Fig. 7.2 Use of sawn timber and plywood lateral bracing (*left*) and engineered glued laminated timber (*right*).

parallel strand lumber, glued laminated timber (glulam) and cross-laminated timber (CLT), amongst others. These are discussed further in Chapter 8.

- Wood composites classified as polymer-wood and polymers-concrete, see Asif (2009).

Fig. 7.2 illustrates typical examples of sawn and EWPs in use.

In structural applications, the reliability and durability of timber products affect their service life, which ultimately impacts not only sustainability but also the amount of human activity required if frequent maintenance and/or reconstruction is necessary.

7.3.2 Structural reliability

7.3.2.1 Tree structure and growth

Trees consist of six main layers: the outer bark, the inner bark, the cambium, sapwood, heartwood and the heart or pith. The heartwood is the mature part of the tree, consisting of dead cells filled with tannin and extractives deposited during the tree's growth. These deposits provide the unique colours seen in the different species of timber; they also provide certain levels of natural durability. The sapwood layer comprises living, open cells which transport water and salts between the ground and the leaves. Sapwood is lighter and has little natural durability (when used externally in building products this wood is generally preservative treated).

From an engineering perspective, being a natural material, timber has considerable variability in its engineering strength properties related to the growth patterns within the trees and the microstructure of wood itself. Tree growth involves the formation of new cells in the cambium layer just below the outer bark. In temperate regions this growth is most active in the spring and early summer and forms low-density 'early wood'. In the late summer, slower growing and winter higher density 'late wood' forms that is stronger than early wood. Early and late wood forms characteristic growth rings that are visible to the naked eye. In tropical regions this growth pattern is not present. Other growth-related factors include the differentiation between the sapwood and heartwood (Fig. 7.3), but this only affects durability; for a fixed moisture content sapwood and heartwood have approximately the same strength. At the microstructure level wood consists of elongated cells (0.7–0.8 mm), which align themselves more or

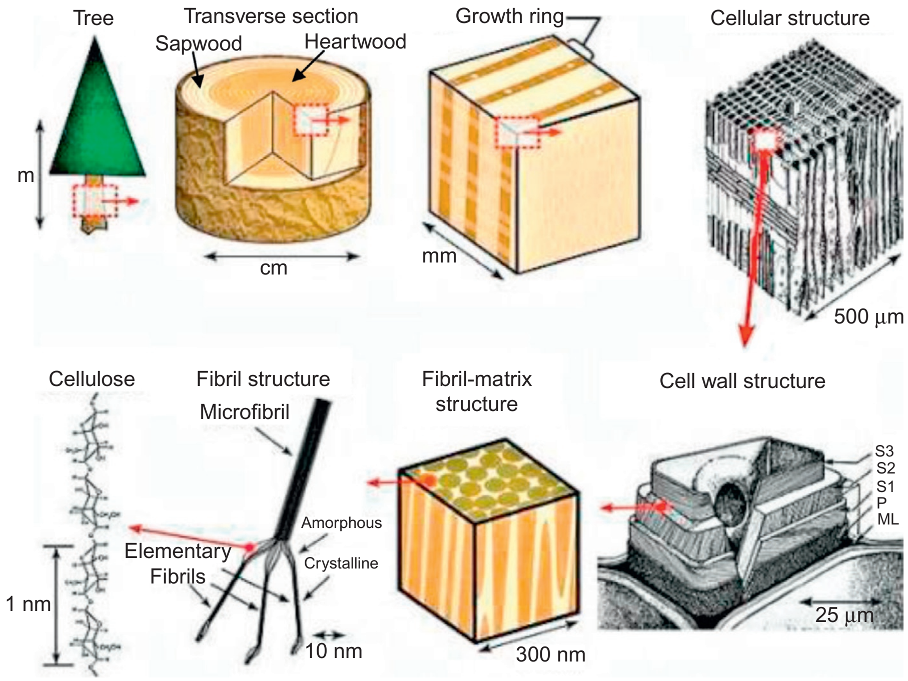

Fig. 7.3 Growth pattern and wood at various size levels.
From www.gizmag.com/cellulose-nanocrystals-stronger-carbon-fiber-kevlar/23959/.

less parallel to the tree trunk and these cells are linked in the transverse direction by hydrogen bonds and what are known as ray parenchyma.

The microstructure details matter less than the fact that wood is much stronger in the trunk direction (parallel to the grain) than in either the tangential or radial (perpendicular to the grain) directions; in engineering terms it has orthotropic strength properties. Recall the simple fact that wood is easily split with an axe in the trunk direction but is much more difficult to cut transversely. Tree branches are also strong in the branch direction and are at an angle of approximately 90 degrees to the trunk axis. Branches becoming enclosed by the trunk can interfere with the direction of the wood fibres in the trunk and cause localised deviations of the trunk cells at an angle that can be quite large, so that it has heterogeneous strength and stiffness material properties. Other phenomena such as spiral grain can also change the orientation of trunk wood fibres.

7.3.2.2 Sawn timber

Sawn timber is generally processed using longitudinal saw cuts parallel to the trunk axis, which means that the strong direction is not necessarily aligned with the longitudinal axis of the sawn timber. Theories of failure for engineering materials make it clear that, when the wood grain is at an angle to the sawn timber axis, sawn timber strength in its longitudinal axis direction is reduced. Greater grain angles cause greater strength losses. At knots the disturbance of grain angle is substantial. Grain angle disturbance means that individual pieces of timber will have widely differing strengths even when coming from the same tree trunk, which has resulted in sawn timber being marketed in strength grades that can cover a wide range of strength values. Designs are based on lower 5th percentile strengths meaning that the majority of pieces can be working at load levels well below their capacity; this is unavoidable if adequate safety margins are to be maintained.

7.3.2.3 Timber properties and moisture content

Sawn timber is graded, or sorted, for strength and stiffness using either visual or mechanical methods. With visual grading, grain angle relative to the timber longitudinal axis and the sizes of knots (enclosed branches) are the principal determinants of grade; with machine grading a relationship between modulus of elasticity and strength values is the principal but not the only determinant. Neither of these methods can accurately predict the strength properties; consequently extensive testing is required before statistically accurate strength and stiffness values can be assigned. Once the sorting is complete, appropriate grade marks are placed on individual pieces (the marks differ between countries) which relate to a set of defined characteristic properties for bending, strength and stiffness as well as compression, tension and shear strengths. The procedures are species dependent in general but are applied across groups of species in some instances.

Structural timber can be either used ‘green’ (unseasoned) or, what is more common today, most commercial timber products are ‘dried’ (seasoned) using a mixture of air-drying and kiln-drying technologies. Drying timber increases its stability, strength, stiffness and nail-holding capacity. Timber is generally dried to a moisture content in equilibrium with its service environment (used externally 10–18%, for internal appearance grade products 9–14%). Softwoods have quite a porous cell structure and can be dried quite quickly (weeks). Hardwoods, on the other hand, have a nonporous cell structure which makes drying very slow (over 12 months for some species). Moisture content is simply the mass of water to mass of solid wood expressed as a percentage.

7.4 Durability

7.4.1 Introduction

In the context of sustainable construction, a key specification issue is durability performance. Appropriate species used under appropriate environmental conditions can provide outstanding performance. The Horyuji Temple in Nara, Japan (Fig. 7.1), mentioned earlier, was built using Japanese cypress or hinoki which is highly resistant to decay because of its high resin content, but its longevity was enhanced by protecting the major components from moisture. Also many of the buildings in Venice including the bell tower in St. Mark’s Square have foundations of timber piles of oak and pine driven into a marshy swamp. The bell tower foundations can be traced back to the AD 900s and have survived because of the lack of oxygen in the swamp soil. Whilst these examples all illustrate the potential longevity of wood, this is not the norm.

7.4.2 Decay mechanisms

According to Asif (2009), it is possible to categorize decay mechanisms as one of two types:

1. biotic decay arises where wood is attacked by bacteria, fungi, insects or marine borers,
2. abiotic decay caused by heat, oxygen, moisture (rain, humidity and snow), polluting elements and chemicals and sunlight (ultraviolet).

Biotic decay in a terrestrial setting is most commonly caused by fungi and insect attack with bacteria having little impact on wood, USDA Forest Service, Forest Products Laboratory (2010).

7.4.2.1 Fungal decay

Fungal decay typically requires three conditions:

1. wood moisture content >20%,
2. the presence of oxygen,
3. temperatures in the range 25–40°C,

Sapwood is more susceptible to fungal attack than heartwood because of the higher concentration of sugars and carbohydrates. The fungi rely on these as their food source, which means that timber should be preservative treated if it is used externally. Fortunately, sapwood can easily be impregnated with preservatives although it is more difficult to preservative-treat the heartwood. On the other hand, heartwood of different species has varying natural resistance to fungal decay. Commercial timber species have all been assessed and allocated a specific Durability Class for external design; this allows particular timbers to be specified for 'outside above ground' use (decking, sidings, etc.) or 'in-ground' use, with confidence that they will last an expected number of years.

Good design detailing is also important in keeping the wood dry. This includes the use of roof eave overhangs and verandahs, shielding or capping of exposed beams, or providing a top surface at an angle such that moisture naturally sheds and has little opportunity to penetrate the wood, bearing in mind that moisture diffusion through wood is a slow process. Full details are provided by MacKenzie et al. (2007), including estimates of service life for different construction elements.

With care, timber houses can be expected to resist fungal decay for at least the best part of a human lifetime with components within the building envelope protected from the elements expected to last for centuries.

7.4.2.2 Insect attack

Termites are present in a wide variety of climates and occur in all continents except Antarctica. If they invade a susceptible timber structure they can be extremely destructive. According to Bootle (2005) there are two main types: subterranean and drywood termites. The subterranean species rely on having ground contact and therefore are somewhat simpler to detect in view of the tunnels that they build from the ground to the timber under attack (many of the termite protection treatments rely on regular visual inspection). Drywood termites can survive without ground contact by living off the moisture contained within the wood but require high humidity such as found in tropical and subtropical climates. Drywood termites are more difficult to detect. Terrestrial insects other than termites include borers and beetles. These are usually less of a problem than termites. In marine environments the Toredo borer can pose serious problems. Recommendations on service life considerations for dealing with insect attack are provided by Mackenzie et al. (2007).

7.4.3 Abiotic degradation

Asif (2009) makes the point that "heat, oxygen, moisture (rain, humidity and snow), polluting elements and chemicals, and sunlight (ultraviolet) have the tendency to adversely affect the service life of wood especially in outdoor applications. Unfinished, unprotected timber will inevitably weather as a result of gradual changes to its physio-chemical structure brought about by temperature and moisture content variations. This degradation process, also called weathering, starts at the surface of the wood through photo-oxidation of the surface catalyzed by heat and ultraviolet (UV) radiation in sunlight". The weathering process leads to a breaking down and wearing away of surface fibers, change in colour (silvery-grey) and roughening of the surface (the rate, though, is generally slow: exterior approximately 3–6 mm per century). It is less of a problem from a safety point of view relative to fungal decay and insect attack; its role in decay is mainly associated with cracking providing a possible pathway for fungal attack.

7.4.4 Preservation and timber finishes

7.4.4.1 Fungal and insect attack

A number of preservation measures are available that can extend the life of structural timber. The sapwood in the tree has no natural durability and is the most vulnerable, but fortunately it can, with relative ease, be infused with chemicals to provide protection. Timbers can be treated to a number of different levels depending on the exposure environment (inside use; outside use above ground, in ground, in marine water) and the biological hazard to be protected against (insects or fungal decay). A wide range of timber preservative treatments is available including: fixed water-borne preservatives such as copper-chrome-arsenate (CCA), alkaline copper quaternary, copper azole; light organic solvent preservatives; and oil-type preservatives such as creosotes. CCA treatments are still utilised in many countries but usually restricted to products where there is no ongoing hand contact, such as deck joists and bearers, etc., and not used in products such as playground equipment, picnic tables, hand-rails, decking boards, garden furniture and exterior seating. CCA is also used extensively for farm fence posts and extends their life from a few years (<10) to 40 years or more. It is effective against fungal attack and insect attack. Bootle (2005) outlines the different preservative possibilities and includes some details of treatment requirements.

7.4.4.2 Weathering

There are two primary methods for protecting timber against weathering:

- paints, water repellents, water-repellent preservatives and pigmented penetrating stains,
- protective coverings—simply not exposing timber to UV light and rain.

When choosing a finish it is generally a matter of deciding on an appropriate balance between maximum service life (frequency of maintenance) and maximum naturalness in appearance. Paints generally provide the longest time between maintenance periods, but tend to obscure the timber's natural grain and colour, while stains allow the grain to remain visible, but maintenance periods tend to be shorter, 3 to 6 years. Shanks and Nolan (2012) provides detailed information on selection of finishes, systems and maintenance.

Timber does not differ from other materials in respect to needing ongoing maintenance or protection from the weather. Typically steel exposed to external conditions requires periodical repainting.

7.4.5 Fire

Surprisingly to many, large timber sections actually resist fires in an efficient and predictable manner. In a fire, a char layer forms on the outer surface which protects the inner parts from reaching excessively high temperatures. This charring occurs at a known rate depending on species density, and accordingly large timber beams can be designed to resist potential fire loads by adding sacrificial charring layers above the required structural member size. Steel, by comparison, is an excellent heat conductor and in fact loses strength and stiffness at around 500°C when fires typically reach temperatures of 800°C or more. In addition to natural charring, modern timber structures that require fire ratings today also use a combination of encapsulation, using fire-rated plasterboard and fire sprinkler systems.

7.5 Material environmental assessment

7.5.1 Introduction

The building and construction sector was an early mover in embracing environmental change in many parts of the world both through minimum government regulations and voluntary market approaches. The green building councils around the world have been very influential in having the business case for green buildings recognised by the property sectors and governments in their respective countries and, as such, green buildings are now in demand from developers and owners. A wide range of international environmental assessment and rating tools has also been developed over this time, focussing on different issues and metrics; some have had a single-issue focus such as recycling, whilst others are restricted to the energy efficiency of heating and cooling during the operation of a building. A number take a broader approach, incorporating a range of planning-related issues, and some take a comprehensive LCA focus.

Reed et al. (2011) notes that many of these tools differ considerably with regard to what they actually assess, how they operate and whether they can be compared directly with assessment tools from other countries.

While it is recognised that different tools are developed for different users and uses, it is also clear that there is a need for the adoption of a consistent basis and methodology, particularly in building and construction materials assessment. In this regard a consistent view globally is that LCA offers the most scientific and accurate evaluation of environmental impacts of materials, assemblies or whole buildings over their entire lives. The World Green Building Council now also accepts this position. In its report 'The Business Case for Green Buildings' (GBCA, 2013) it states that:

> *the first wave of high-performing, green building arose as a response to demand for energy and resource efficiency. Times have changed, and strengthened by the advent of green building rating tools, the industry now recognizes that green buildings deliver much more than energy efficiency alone. Buildings must be examined in the context of their impact on the local, natural and built environments - and their neighbours.*

In terms of actual materials, the report also states that:

> *a material is usually considered in terms of its entire life cycle, as well as its attributes at the time of installation, utilizing a cradle-to cradle approach that takes into consideration the embodied energy, toxicity and emissions, replacement cycles and disposal to ensure that a material is 'green' in all aspects.*

7.5.2 LCA process

LCA provides a detailed formal process of quantifying the environmental effects of a product, or a building, throughout its entire life (cradle to grave). It accounts for all the

- material and energy usage (*inputs*), and
- subsequent environmental impacts (*outputs*).

The development of the ISO 14040-14043 series of standards has placed LCA on a disciplined footing. The general framework is illustrated in Fig. 7.4.

In Europe, a harmonized approach is now being taken through the European Committee for Standardization (CEN) TC350, and a number of key integrated building performance standards are now in use including: *EN15643-2 Framework for Environmental Performance, EN15978 Assessment of Environmental Performance and EN15804 Environmental Product Declaration*s. These also form the basis for reference documents of LCAs undertaken in many countries globally.

According to EN15804, the following life-cycle stages for building assessment are recognised:

- *A1–A3 Product* (raw material supply, transport, manufacturing),
- *A4–A5 Construction* process (transport, construction installation process),
- *B1–B7 Use* (use, maintenance, repair, replacement, refurbishment, operational energy use, operational water use), and
- *C1–C4 End of Life* (deconstruction, demolition, transport, waste processing, disposal)

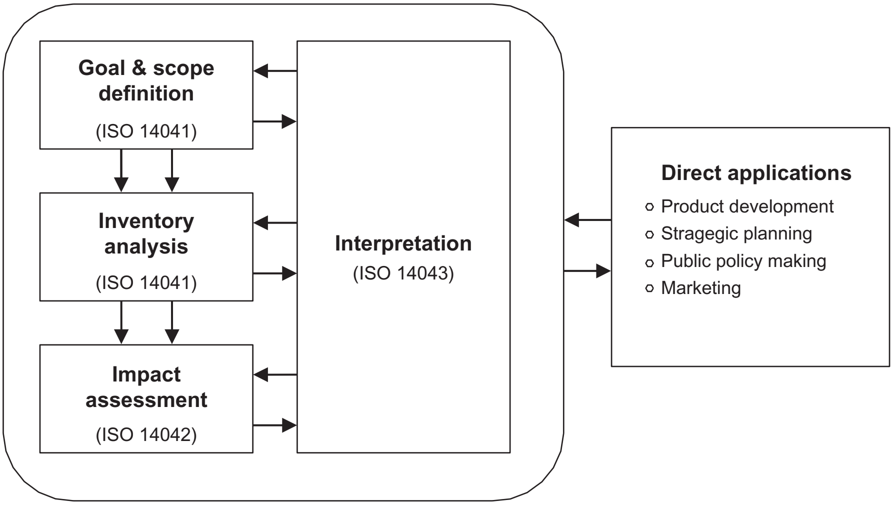

Fig. 7.4 Life-cycle assessment methodology ISO 14040 framework.

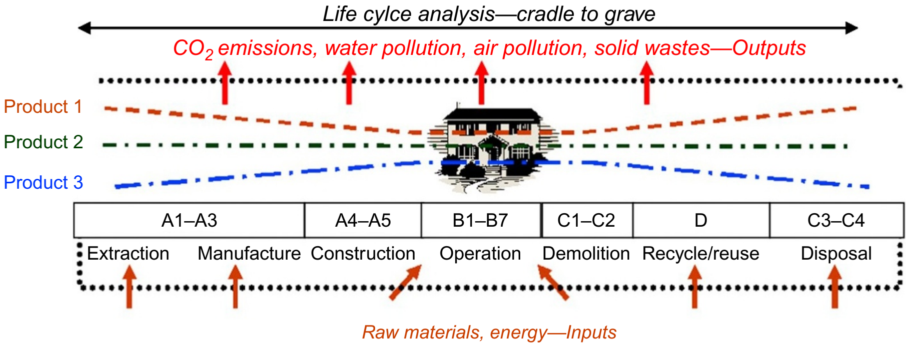

Fig. 7.5 Typical life-cycle stages for a building.

Also, for optional inclusion:

- *D Benefits and Loads Beyond the System Boundary* (reuse, recovery, recycling potential)

Fig. 7.5 illustrates these stages in terms of the overall life of a building constructed of multiple products.

When using LCA to assess the environmental impacts of materials or structures, the boundary conditions and functional unit used in the study are critical if a fair comparison between materials is to be made. LCA studies may assume a full *cradle to grave* approach or a more specific *cradle to gate* focus. The gate, the end point of the analysis chosen by the analyst, is typically, for example, completion of a manufactured product, a constructed house of multiple products, a home at end-of-life, that is, after X number of years of operation, or a full LCA.

7.5.3 Important considerations with LCA

Key to any LCA are the considerations and positions taken around:

- functional unit (the agreed definition that allows objective comparison),
- system boundaries (the limits of what flows are included in the analysis),
- primary products and coproducts,
- allocation approaches (the process of attributing impacts or benefits to a particular part of a process that results in multiple outputs or coproducts),
- inventory data collection,
- analysis assumptions,
- analyses, and
- results evaluation and interpretation.

For forest and wood products, Perez-Garcia et al. (2005) suggest that there is greater complexity of environmental analysis including:

- the time for forest growth and the long lifespan of some wood products,
- the wide range of useful products that are obtained at different points in time, including forest thinning during the time of forest growth, primary products and coproducts at the time of forest harvest, and combustible residues at the end of the product lifespan,
- the broad array of joint products that can be obtained from a tree (eg, saw, veneer and pulp logs) and a stand (eg, different uses from different species for a mixed forest stand), and
- the unique relationship between forest development and environmental services.

7.5.3.1 Function and functional unit

In a structural context a function refers to the structural role of a building component, for example, supporting a specified load, if load bearing, or simply dividing off one area from another if non-load bearing. The functional unit is a quantitative measure of the environmental impact of the component or system of components that references inputs and outputs in comparative LCA analysis. With timber product assessments, commonly used units often include material mass, length or volume. Typical examples follow.

- For forests: 1 m^3 of log of a specific grade (May et al., 2009).
- For sawn timber (lumber): 1 m^3 of planed kiln-dried softwood lumber at an average density of 433 kg/m^3 (AWC, 2013).
- For plywood (panel product): 1 m^2 of 12-mm-thick softwood, 5% moisture content (dry basis), average product density of 491 kg/m^3 (Wood for Good, 2015).
- For an engineered I-beam: 1 km of I-beam product output that includes harvesting and forest management activities, transportation of logs, Canadian lumber, OSB, and LVL manufacture, delivery of inputs to the I-beam facility and the production of I-beams (Athena, 2013).

With comparative analysis, the mass/length/volume approach is usually not appropriate when comparing two different material products where, for example, 1 tonne, or 1 m^3 of timber does not usually fulfil the same structural function as one tonne, or 1 m^3, of concrete. It is more appropriate in these instances to compare products based on a common function such as 1 m^2 of load-bearing dividing wall. Large-scale buildings constructed to a unique design with a varying mix of materials can only be compared by separate analysis of design alternatives using a different mix of components.

7.5.3.2 Allocation

Allocation is the practice of attributing impacts or benefits to a specific part of a process that results in multiple outputs or coproducts. Allocation is somewhat of a contentious area within LCA circles, particularly as it is a subjective process and can be influenced by the personal views and values of the assessor. Different approaches can be used including 'economic' or 'mass' allocation, that, in turn, provide different results based on the assumptions used; in some cases it may be beneficial to carry out both analyses to assess the different results. Allocation is particularly significant for wood as multiple coproducts of varying values and volumes can be produced from the one raw material, and these wood products themselves at the end of their service life can be used as biofuels for cogeneration of energy. It is generally accepted that, wherever possible, allocation issues should be avoided either by dividing the processes and using more detailed data, or by system expansion, adding additional functions to the functional unit so the systems compared have identical functions.

Typically for most timber products a mass allocation method is used. The AWC (2013) EPD for North American Softwood Lumber, for example, states the following. "Allocation of multi-output processes should be based on mass. However, if economic value difference is at least ten times greater between products from a multi output process, a suitable revenue based allocation principles shall be applied and these deviations shall be substantiated and readily available for review". The lumber coproducts fall within this 10 times value threshold and were thus allocated on a mass basis.

7.5.3.3 System boundaries

The system boundaries define the limits of what flows are included within the specific LCA analysis. For forest-based products (and other biological materials) that can be harvested and regrown, the system boundary is somewhat unique compared to other building products (such as aluminium, steel, concrete or masonry), which generally come from the mining of existing finite resources. Trees can be grown and harvested in relatively short time frames so the growth phase can be included within the system boundary. For sawn timber harvest, rotations are around 30 years for softwood, and around 60–80 years for hardwood; for some applications such as paper making and packaging products rotations can be much shorter. New trees can then be grown, providing a sustainably managed process for production. In this instance the system boundary needs to include forest operation impacts such as planting, fertilizing, thinning and ongoing management prior to the extraction process and also as part of this phase the carbon sequestration in the trees. Fertilizing may or may not be required depending on the site soil conditions or plantation rotation periods to maintain wood production.

For timber products, detailed life-cycle inventory (LCI) data collection has been undertaken over the past decade in a range of countries: in Australia (May et al., 2009), in North America (CORRIM), in the UK (BRE), in Europe (multiple countries). In terms of gathering of this LCI data, the system boundaries for the LCAs are generally undertaken for different specific segments of the wood product production chain as shown in Fig. 7.6.

Results of LCAs for different production segments can be aggregated to give total production chain results. For instance, for sawn wood the system boundary would include the forest segment and the sawmill segment as illustrated in Fig. 7.7 (AWC, 2013).

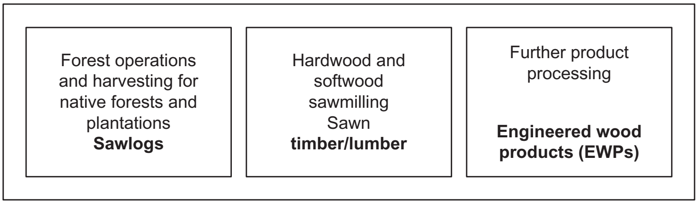

Fig. 7.6 Typical system boundaries for determining timber LCI data.

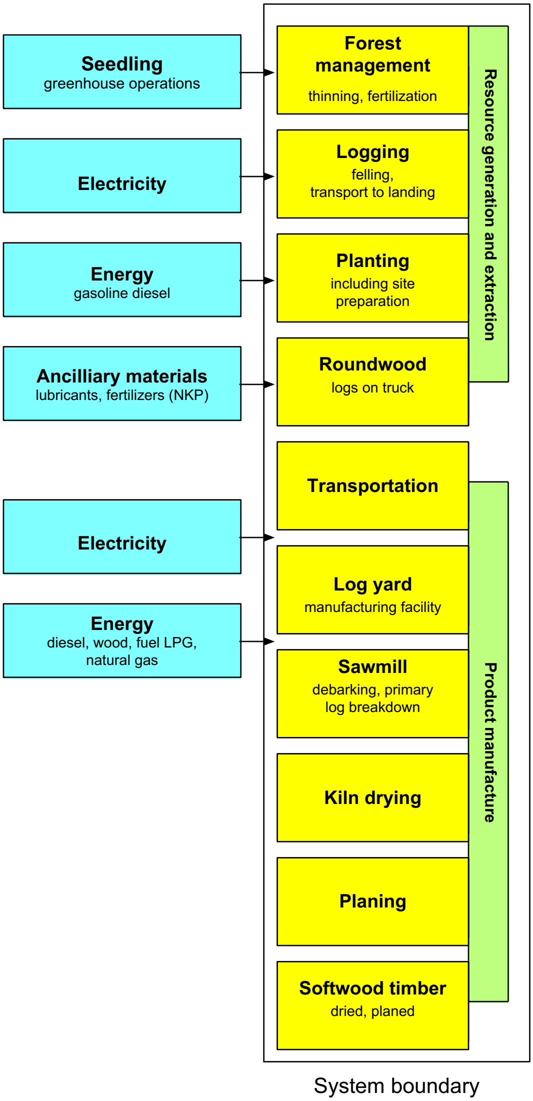

Fig. 7.7 System boundary for sawn softwood lumber.
Courtesy of American Wood Council, Leesburg, VA.

7.5.3.4 Environmental product declarations

From the detailed LCA studies of the product segments, simplified environmental product declarations (EPDs) are produced that summarise the LCA findings and allow them to be used by LCA practitioners. The availability of timber EPDs is now becoming much more common. See

- UK: http://woodforgood.com/lifecycle-database/
- US: http://www.awc.org/greenbuilding/epd.php
- Canada: http://www.athenasmi.org/resources/publications/
- Australia and NZ: http://www.epd-australasia.com/
- Europe has multiple schemes but the biggest is IBU: http://construction-environment.com/hp2/Institut-Bauen-und-Umwelt-e-V.htm

7.6 Wood construction and LCA

Each of the main life-cycle stages will now be examined in regard to timber building products and compared against possible alterative materials.

7.6.1 Product manufacture (stage A1–A3)

This is the first phase of the process ‘from cradle to the manufacturing gate’ and includes all impacts due to raw material supply, transport from the extraction point to the manufacturing facility and the physical manufacture.

Recognising that global warming is a major driver for the current high global awareness and interest in environmental improvement, of particular importance with wood products is the impact of product manufacture on CO_2 emissions. There are a number of mechanisms by which using wood products, particularly as a substitution for other existing construction products, affects global greenhouse gas (GHG) balances. These include the following:

- carbon storage in the forest,
- carbon storage in wood products,
- avoided alternative material process emissions,
- less fossil fuel consumption in manufacturing (embodied energy and carbon),
- avoided fossil fuel emissions due to biomass substitutions,
- carbon dynamics in landfills.

7.6.1.1 Carbon storage in the forest (sequestered carbon)

Growing forests absorb carbon dioxide from the air, and using the solar energy of the sun through the process of photosynthesis, break down the CO_2 into carbon which is sequestered (stored or fixed) in the tree biomass (wood, leaves, roots) and soil, whilst the oxygen is released back to the atmosphere. Trees are the most powerful concentrators of carbon on earth. Every 1 kg of log contains sufficient carbon to produce or remove 1.65 kg of CO_2; this computation is based on the fact that dry wood is 45%

carbon and, based on atomic masses, CO_2 contains 27% carbon. Through photosynthesis, carbon is concentrated in

- *hardwood trees* (densities 650–900 kg/m^3), around 1.8–2.6 million times producing wood that contains 290–405 kg of carbon per cubic metre (1.2–1.65 tonnes CO_2/m^3);
- *softwood trees* (density approximately 500 kg/m^3), around 1.4 million times, producing wood that contains about 250 kg of carbon per cubic metre 0.92 tonnes CO_2/m^3 (Moore, 2000).

Young forests grow vigorously, adding to the stored carbon pool. At some point in a forest's life the carbon being stored equals the carbon being lost through forest litter decomposition and then the mature forest moves into a senescent phase, whereafter the trees start to die and decompose. A forest left naturally to grow and die (or lost through fire) is effectively carbon neutral with any carbon being sequestered later being released to the atmosphere where new growing trees reabsorb the carbon and the cycle continues.

7.6.1.2 Carbon storage in wood products

If the forest is a sustainably managed production forest and trees are harvested and regrown, then this provides a different equation. When trees are harvested for wood products, this is not a carbon emission but a carbon transfer from one carbon pool—the forest—to another, the products. Sathre and O'Connor (2010) uses the analogy of transferring money between bank accounts; the money is still in the bank, just in another account. The harvesting and use of the timber in comparatively long-life products, such as building components, furniture, etc., ensures that the carbon dioxide remains 'fixed' for long periods. Continually replacing the harvested trees with actively growing trees ensures that the net sequestration of carbon dioxide continues. A typical North American single-family home stores about 9.3 tonnes of carbon (34 tonnes of CO_2) in the building structure (Dovetail, 2014). In the short to medium term, significant climate benefits could result from substituting wood-based products for metal- and concrete-based products or using longer lived wood products; however, in the long term this benefit will be subject to diminishing returns with the new wood products entering construction balancing the used wood leaving.

7.6.1.3 Avoided alternative material process emissions

Sathre and O'Connor (2010), however, also points out that "the carbon bank balance goes up significantly when an additional carbon pool is considered: the avoided emissions due to wood use in place of other materials" (such as steel, concrete, aluminium, etc.). A wide range of research studies have examined this substitution impact with many presenting views on a 'displacement factor', effectively a measure of the amount of carbon emission avoided by the use of wood instead of some other material with a higher embodied energy/carbon impact. Sathre and O'Connor (2010) examined and discussed data from 21 studies. He observed that a relatively large variation in results existed based on the different system boundary assumptions adopted. Regardless, this metaanalysis found that: "all of the studies reviewed found that the production of wood based materials and products results in less GHG emission than the production of functionally comparable non-wood materials and products".

7.6.1.4 Manufacturing: embodied energy and net carbon impacts

The embodied energy and net carbon impacts of a building material cover all life stages including its raw material extraction, product processing, transportation, construction, use/maintenance and disposal or reuse. The embodied impact of the materials phase has been shown to be significant in a full LCA sense. Carre (2011) reported that it could be as high as 51% in temperate climate zones, and in the cooler climate zones may range from 17% to 29%.

Embodied energy and carbon are two of the main metrics used for material comparison. Every material used in construction contains embodied energy. Manufacturing high-energy materials generates considerable GHG emissions. A comparison of the CO_2 production of different typical building materials including the carbon sink effect is provided in Fig. 7.8.

It should be noted that this approach of comparing CO_2 production per cubic metre of material is of limited value in that it takes no account of the amounts of material required to complete a specific task. Rather than a simple direct comparison of materials it is more appropriate, as discussed earlier, to use a common functional unit; This might be a specific system such as floor assembly, a clad and lined wall or a roof assembly. Embodied energies for typical US residential wall systems are given in Fig. 7.9. In general, what can be noted here is that the higher the proportion of timber involved the lower the embodied energy.

A number of organisations and reports in recent years have provided case studies of projects and the embodied carbon benefits of using wood based on the volume of wood and how much carbon it stores, and the greenhouse gas emissions avoided by using wood instead of steel or concrete. Table 7.1 provides a summary of some of these case studies.

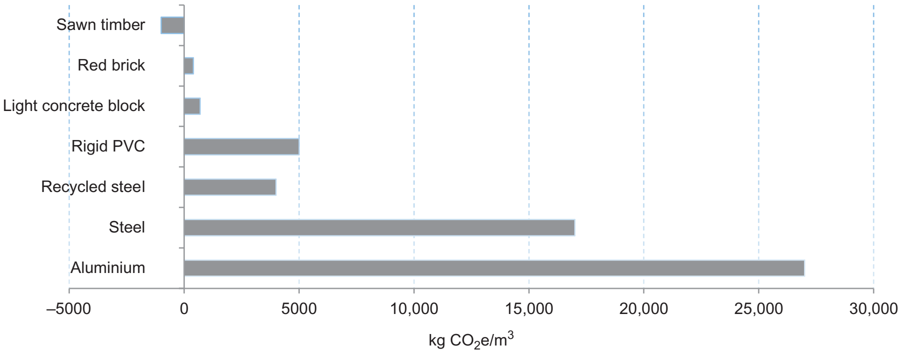

Fig. 7.8 Comparison of the CO_2 production of different typical building materials including the carbon sink effect.
From CEI-Bois (European Confederation of Woodworking Industries), 2011. Tackle Climate Change: Use wood, Building Information Foundation RTS. http://www.cei-bois.org/files/FINAL_-_BoA_-_EN_-_2011_text_and_cover.pdf.

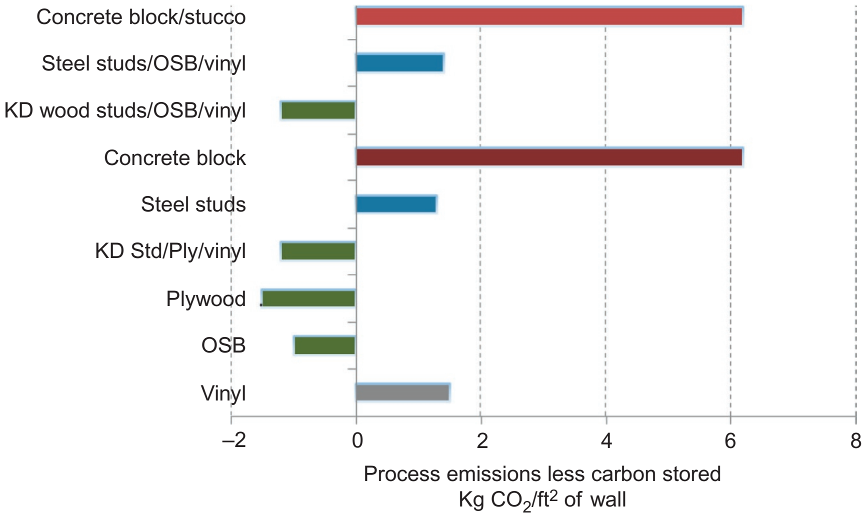

Fig. 7.9 Net product carbon emissions: wall structure (kg CO_2/ft^2 of wall) (Dovetail, 2014). *KD*: kiln dried; *OSB*: oriented strand board.
From Lippke, B., Edmonds, L., 2009. Life Cycle Assessments of Subassemblies Evaluated at the Component Level. Consortium for Research on Renewable Industrial Materials. http://www.corrim.org/pubs/reports/2010/phase2/Module_I.pdf.

7.6.2 Construction process (stage A4–A5)

This phase includes transport of the manufactured product from its manufacturing point through a supply chain to the construction site where installation takes place. As with any other material, the impact here to a great degree will depend on the local availability of the product. In most countries there is a supply of local timber for a range of products but often also a reliance on imports to meet market demand for other specific nonlocally produced products or species.

7.6.3 Use or operational phase (stage B1–B3)

This phase covers the operational phase of the building however long that might be—20, 50, 100 years. It includes all impacts during the use phase of that building including maintenance, repair, replacement, refurbishment, operational energy use, operational water use.

When it comes to materials, the importance in the operational phase involves the construction and performance of the building envelope. Timber-framed and clad systems are regularly used as the construction material in insulated lightweight wall systems which are used extensively throughout Europe and North America in both cold climate zones, to keep heat in, and warm climate zones, to keep heat out. The Canadian Wood Council (CWC, 2009) advises that “Wood is 400 times better than steel and ten times better than concrete in resisting the flow of heat. This means more insulation is needed for steel and concrete to achieve the same performance as with wood framing”.

Table 7.1 Summary of international projects constructed from wood, carbon sequestered and emissions avoided

Project	Country	Vol of wood used (m^3)	Carbon dioxide sequestered in wood (tonnes CO_2)	Avoided GHG emissions of alternative materials (tonnes CO_2)	Total emissions benefit (tonnes CO_2)	Info source
Raised Floor Home	New Orleans, US	15	30	60	90	McCloskey (2009)
Harada Elementary School	California, US	655	490	990	1480	McCloskey (2009)
Stadthaus Building (9 storeys, 29 units)	London, UK	950	760	320	1080	McCloskey (2009)
Forte Living (10 storeys, 27 units)	Melbourne, Australia	960	792	690	1451	WoodSolutions (2015)
Bridport House (8 storeys, 41 units)	London, UK	1536	1152	892	2044	Eurban (2013)
42 Housing Units	Austria	–	1205	385	1590	CEI Bois
Richmond Olympic Oval	Vancouver, Canada	3820	2940	5880	8820	McCloskey (2009)

Wood-framed windows also provide a more environmentally thermally efficient alternative to more thermally conductive materials. An LCA study on the comparative service life of window systems (Howard et al., 2007) investigated 51 window archetypes made from four different frame materials: standard aluminium (nonthermally broken), PVC, timber and aluminium-skinned timber. Key findings from the report included that "timber framed windows, whether aluminium skinned or without aluminium skin had consistently lower embodied impact per m^2 of window expressed either in terms of embodied CO_2 or eco-points – initial, over the life or for energy implications".

Maintenance, repair and replacement are also key factors in the operational phase and of particular importance to wood-based products used externally, such as cladding (siding), decking, windows, etc. The key here is proper selection of timber species of appropriate durability class (discussed previously).

7.6.4 End of life (stage C1–C4)

Once the building has completed its service life it will be either deconstructed, and products will either be reused or recycled (see next phase D), or demolished, and demolition waste will then be transported to a waste processing facility or disposed of generally in landfill.

Wood construction lends itself to somewhat easier deconstruction than other systems such as concrete, particularly when connections are nailed or screwed rather than glued. Many wood products are retained at end of life for either direct reuse or recycling into another wood-based product. If a wood product can no longer be reused or recycled, then one of two things can happen to it. It can either be used as a biofuel for cogeneration of energy, as discussed in the next section, or it can be sequestered in landfill.

If the former process is pursued, then it is important that the storage occurs in conditions where there is no oxygen (anaerobic); in this environment the carbon in the wood will remain permanently sequestered (note: if oxygen exists, then decomposition will occur with a fraction of the wood decomposing to methane which has 22 times the global warming potential of CO_2; in some well-managed landfills methane is captured and used as an energy source, but not in all landfills). The long-term sequestration and decomposition of carbon inherent within wood when deposited in landfill has been the subject of much research in the last decade (Ximenes et al., 2008; Wang et al., 2011). Results have found that carbon decomposition and methane emissions from landfills were considerably lower than previously thought. The findings from these studies have indicated that default values by the Intergovernmental Panel on Climate Change (IPCC) and many governments around the world are extremely conservative.

7.6.5 Benefits and loads beyond the system boundary (stage D)

Though this is an optional inclusion for some building LCAs, in terms of the environmental benefits of materials the three R's—Reuse, Recovery, Recycling potential—are critically important and particularly pertinent to wood products.

7.6.5.1 Reuse of wood products

Reusing wood products saves the energy and resources needed to make a new product and the volumes of material going to landfill.

- Many preused wood products are today highly valued by designers wanting an industrial or used look; structural beams, columns and flooring are in particular demand.
- Secondhand wood building products are valued and regularly used in home renovations to match original products. These include flooring, windows, doors, mouldings, etc.
- Durable external wood products such as hardwood poles are reused in marina, external building applications or landscaping projects.
- Wooden furniture can be used for centuries and passed on from one generation to the next.

7.6.5.2 Recycling

Wood product waste can be either directly or indirectly recycled.

- Direct recycling involves manufacturing a new timber product from what was waste wood. Examples include: finger jointing of small off-cuts into longer sections (often done in-mill for wall studs), glue-laminating timber to form larger products (eg, stair treads), re-milling demolished construction materials (flooring, beams, joists) to make new flooring, parquetry or furniture components. Refer to Chapter 8.
- Indirect recycling involves changing the form of the wood waste to make another product. Hardboard utilises sawmill wastes and forest thinnings. Unrecyclable solid timber can also be chipped for landscape mulch, playground surfacing or compost.

Recycling of timber products is highly energy efficient, particularly compared to alternative building materials such as aluminium and steel.

7.6.5.3 Energy recovery

Wood waste, from harvesting operations and sawmilling processes, and timber products at end of life (those that cannot find a higher value use) can be burnt for energy recovery. The energy generated by this biomass can replace energy generated through the burning of nonrenewable resources. The carbon released during biomass burning returns to the atmosphere where it can again be taken up by new growing trees, thereby ‘closing the carbon loop’—carbon neutral. In LCA terms this involves a ‘cradle-to-cradle’ approach.

As a commercial fuel source, sawmills regularly utilise their sawdust and mill waste to fire boilers that supply energy for kiln drying. Power stations that are fired by nonrenewable energy sources such as coal can also introduce the capacity to co-fire wood waste. From a greenhouse gas and renewable energy source perspective, there is real potential in further developing biomass as an energy source.

7.6.6 LCA case studies on completed buildings

With LCA now a generally accepted process globally there has been an increasing number of studies undertaken on building LCA design throughout the world. These studies have often used different functional units, and also different boundary conditions, but regardless of the different conditions, in virtually all instances the researchers

Table 7.2 Comparison of results from six international LCA studies

Research study (Note: Each study has used different building structures and boundary conditions so direct comparison between studies is inappropriate.)	Timber		Steel		Concrete	
	Cradle to gate energy use (MJ/m^2)	GWP (CO_2-equiv. kg/m^2)	Cradle to gate energy use (MJ/m^2)	GWP (CO_2-equiv. kg/m^2)	Cradle to gate energy use (MJ/m^2)	GWP (CO_2-equiv. kg/m^2)
Trahus Project, Sweden	−530	30			1770	400
Project Athena, Canada	1140	280	1740	340	2520	420
CORRIM, Northern USA	969	207	1604	309		
CORRIM, Southern USA	580	100	810	170		
Lund Institute of Technology, Sweden	680				1900	
Chalmers University of Technology, Sweden	840	40			1430	110

have found that ‘using wood products compared to alternative construction products results in lower environmental burdens’.

A comparison of seven independent case studies undertaken by a range of different institutes in Sweden, Canada and the USA has been undertaken by Eriksson (2004). He concluded that in “all cases, the wood structure results in lower energy use and global warming potential (GWP) than the alternatives, regardless of differing system boundary conditions applied in the different studies”. Results from six of the studies are provided in Table 7.2.

The Canadian Wood Council in their publication ‘Energy and the Environment in Residential Construction’ (CWC date uncertain, see web link) report the results of a study using the Athena LCA tool of three identical homes: one constructed with softwood framing and timber I-beam floor joists, one with light frame steel, and one with insulated concrete forms and slabs. The results found that relative to the wood design, the steel and concrete designs, respectively:

- embody 26% and 57% more energy;
- emit 34% and 81% more greenhouse gases;
- release 24% and 47% more air pollution;

- discharge 4 and 3.5 times more water pollution;
- use 11% and 81% more resources from a weighted resource use perspective; and
- produce 8% and 23% more solid wastes.

Sathre and O'Connor (2010) has provided a synthesis of 66 research studies (English, Swedish and Finnish) investigating wood products and greenhouse gas impacts; at least two-thirds of these involve comparative studies of wood and alternative construction materials or products. He concludes that "the studies cover a wide range of wood product types and materials substituted, use data specific to different geographic regions, and employ different methodological techniques and assumptions. Collectively, however, the 21 studies provide a consensus that wood product substitution reduces GHG emissions".

A major Australian study (Carre, 2011) compared the life-cycle impacts of a 203-m^2, three-bedroom, 6 Star energy-rated house, constructed using five different construction variations involving insulated steel and timber wall and floor frames, concrete slab-on-ground, and brick veneer and lightweight timber cladding. Key findings from the study included the following (FWPA, 2011).

- "Substituting wood products from well managed forests and plantations for more greenhouse gas (GHG) intensive building products in cladding, wall, roof and floor framing could reduce the GHG emissions of a typical house by up to 18 tonnes over its life.
- The biggest GHG reductions can be made by;
  - substituting timber cladding for brick veneer,
  - using timber for wall and roof framing instead of steel,
  - using timber for a suspended floor instead of either steel or concrete slab on ground.
- Up to 51% reduction can be made using wood products"; see Fig. 7.10.

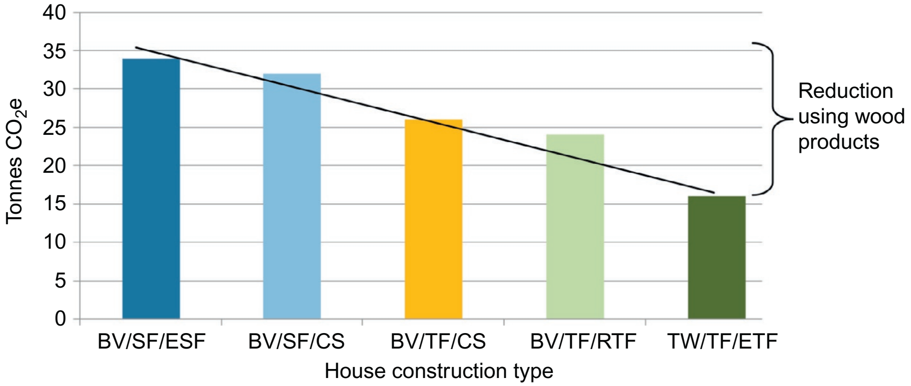

Fig. 7.10 Embodied greenhouse gas emissions of house construction systems (Melbourne). BV, brick veneer; *SF*, steel wall frame; *RFS*, elevated steel floor frame; *TF*, timber frame; *CS*, concrete slab-on-ground; *ETF*, elevated timber frame floor.
From Carre, A., 2011. A Comparative Life Cycle Assessment of alternative constructions of a typical Australian house design. http://www.fwpa.com.au/rd-and-e/market-access/204-comparative-life-cycle-assessment-of-alternative-constructions-of-a-typical-australian-house-design.html.

7.7 Future trends

With the accelerated transformation towards low-carbon and resource-productive economies, and the continuing interest and desire of designers and consumers to use more environmentally friendly materials, the future for wood and timber products seems particularly positive. No other major building material can be sustainably renewed, produced with such low-embodied environmental impact, nor have such a range of opportunities at end of life (reusing, recycling or recovering energy).

The increased innovation and investment in new wood-based products, particularly the growing range of EWPs, will no doubt see a wide range of additional timber construction products come into the market over the next decade. Hybrid softwood and hardwood CLT panels engineered to maximise performance or look; thermally and chemically treated products with improved strength and durability, composite concrete and timber systems, and posttensioned timer frames (particularly useful in seismic areas) are all gathering a foothold in different countries around the world.

There are clear benefits in using timber construction not just in environmental terms but also economically with quicker build times, lower site impacts and exciting prefabrication opportunities. Demand continues to grow, and why not? Wood is one of the most desired, useful and environmentally friendly construction materials available and locally available almost anywhere throughout the world.

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Useful Web Links

Australia

WoodSolutions: http://www.woodsolutions.com.au/.
Building Products Innovation Council: http://www.bpic.asn.au/.

Europe

Cei-bois: http://www.cei-bois.org/.

North America: Canada and US

American Wood Council: http://www.awc.org/index.php.
Canadian Wood Council: http://cwc.ca/.
US WoodWorks: http://www.woodworks.org/.
Canada WoodWorks: http://wood-works.ca/.
Naturally Wood (Canada): http://www.naturallywood.com/.
CORRIM: http://www.corrim.org/index.asp.
Athena: http://www.athenasmi.org/.

UK

Wood for Good: http://www.woodforgood.com/.
TRADA: http://www.trada.co.uk/.
Envest: http://envest2.bre.co.uk/.

EPDs

Australia: http://www.epd-australasia.com/.
Canada and US Athena: http://www.athenasmi.org/resources/publications/.
Canadian Wood Council: http://cwc.ca/green/life-cycle-and-epds/.
American Wood Council: http://www.awc.org/greenbuilding/epd.php.
UK: http://woodforgood.com/lifecycle-database/.
IBU Germany: http://construction-environment.com/hp4113/Timber.htm.

General Interest

A Short History of Tall Wood Buildings (Arup): http://www.arupconnect.com/2014/08/05/a-short-history-of-tall-wood-buildings/.
How Tall Can We Build in Wood: http://www.alexschreyer.net/engineering/how-tall-can-we-build-in-wood/.

Sustainability of engineered wood products

8

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8.1 Introduction

Chapter 7 dealt with the fundamentals of sustainability issues from the perspective of a wood/timber life-cycle assessment (LCA) analysis. The emphasis was on wood as a natural and renewable greenhouse-positive resource, and timber as a versatile and widely used construction material. It made the point that a significant percentage of wood products come in the form of sawn timber used primarily in domestic housing. It associated sustainability of timber construction not only with raw resource availability (from forests) and impact of manufacture (embodied energy and carbon), but also with durability, structural safety and fire performance. While sawn timber is the major structural component of domestic housing, some smaller and even larger engineered wood products (EWPs) are also used but there is significant application of EWPs in heavier construction such as multistorey mid-rise apartments, commercial, public and industrial buildings. This potential has been retarded to some extent both by regulatory impediments and by lack of education and recognition by engineers and architects, although this is rapidly changing in the light of climate change issues coming to the fore and the environmental friendliness of wood-based products being recognised, appreciated and harnessed. Having said that, it needs to be remembered that sawn timber, used in a suitable construction form, can cover large spaces. One such hangar, constructed in 1933, is located in Mountain View, Santa Clara County, USA. It is in the form of an inverted catenary 60 m high, occupies an area of 32,000 m^2 and housed airships that were used to patrol the Californian coast during World War II. The roof structure consists of bolted arch trusses, a very efficient, lightweight structural form. More recently, EWPs based on adhesive bonding technology have opened the pathway to newer forms of large and taller building construction.

A primary aim of structural engineered timber has always been to use materials more efficiently. With EWPs the wood fibre is used with increased efficiency exemplified by adhesively bonded structural I-beams; see Fig. 8.6. A 360×70 beam, with 40-mm-thick flanges and 10-mm-thick web, involves only one-third the wood fibre of a solid member of the same overall dimensions yet, in bending about its major axis, will carry approximately 60% of the load of its solid equivalent. It is much weaker about its minor axis, but this can be overcome by using structural arrangements in which the loading is in the strong axis direction. A more subtle advantage of EWPs relative to sawn timber

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[#] Retired.

Sustainability of Construction Materials. http://dx.doi.org/10.1016/B978-0-08-100370-1.00008-1

is gained by redistribution of natural defects, especially knots. A knot in sawn timber can seriously downgrade a member's load-carrying capacity, but the laminating effects that are typical of many EWPs improve both their statistical strength characteristics and their efficiency. EWPs constitute only a small fraction of construction timber use in terms of wood fibre consumption so that at current levels of usage they do not pose any problems with respect to resource availability. More critical issues are environmental performance, durability, structural reliability and fire safety.

EWPs comprise interconnected wood-based elements that are used as structural building components. Within this chapter attention will be confined, primarily, to those that involve adhesive bonding, which include:

- panel products such as fibreboard, chipboard, plywood and oriented strand board (OSB);
- beam and column products such as structural I-beams, laminated veneer lumber (LVL), parallel strand lumber (PSL) and glued laminated timber (glulam); and
- more recently a mass timber product known as cross-laminated timber (CLT), which has some features of concrete tilt slab construction but is much more versatile.

Drivers for using progressively more EWPs have included:

- restrictions on harvesting in older native forests which, in the past, provided large-diameter tree boles from which large rectangular structural sections could be cut as opposed to the modern practice of harvesting smaller, plantation trees having relatively short growing cycles resulting in the sawn timber being less mature and unable to provide the physically larger sections,
- a desire by producers to reduce waste in the timber industry and to make greater use and efficiency of the wood fibre available.

Other forms of EWPs such as roof and floor trusses involve nail-plating, but adhesive bonding is, by far, the most common, versatile and cost-effective interconnection technique.

8.2 Comparative performance of EWPs and sawn timber

Sawn timber products are sold in 'stress grades' related to species and strength-reducing characteristics, principally, knot size and the level of sloping grain—the angle that the wood cells make with the timber longitudinal axis. In particular, knots cause local deviations of the grain direction and weakness due to the perpendicular to the grain strength of timber being only one-tenth of its strength parallel to the grain; large knots are especially debilitating. While not all pieces in a production run have the same level of strength-reducing features, it means that many pieces within the same timber grade have variable strength and that some pieces could be designed using markedly higher stress levels if these were identified and marketed separately. Because of practical difficulties this is not possible, so that engineering design values are based on lower fifth-percentile strengths of a species within a grade defined by rules that determine the grade level. Softwoods in particular, because of their knotty nature and other characteristics, exhibit strength coefficients of variation of around 0.4, and this implies wastage of wood fibre implying that approximately 90% of the pieces have strengths 1.5 times greater than their design strengths.

With EWPs, the natural defects are redistributed which reduces coefficients of variation to 0.2 or less allowing the use of higher design stresses and more efficient use of the wood fibre. EWPs exhibit higher strength-to-weight ratios than sawn timber and, additionally,

- a capacity to use the waste material from sawmilling operations and the manufacture of other EWPs (particleboard, medium-density fibreboard (MDF) and hardboard),
- an ability to be reinforced by other materials (limited quantities of steel and exotic fibres including carbon, aramid and glass) to gain even higher efficiency,
- a capacity to use low-grade wood fibre by positioning it where low stress levels will exist in service.

8.3 Environmental performance of EWPs

8.3.1 Overview

The environmental attributes of wood and timber products, particularly from a LCA perspective, are presented in Chapter 7 and as such will not be revisited here in detail. The attributes include the fact that EWPs:

- are produced from a natural and renewable resource—wood, which comes from sustainably managed native forests and plantations where trees, through the process of photosynthesis and using solar energy, absorb and break down CO_2 from the atmosphere, releasing oxygen and storing the carbon in the woody tissue of the tree,
- act as carbon sinks by continuing to store the sequestered carbon for the life of the product (1 m^3 of softwood effectively stores 1 tonne of CO_2),
- are much more effective in utilising the resource to maximise product performance, 'consuming less' being a major platform of sustainability,
- while requiring more energy than sawn timber still use very little energy in their production (embodied energy and carbon) relative to alternative manufactured materials such as steel and aluminium; see Table 8.1,
- at the end of the building's life can be relatively easily 'deconstructed' due to the simple nailed or screwed connections and potentially directly reused,

Table 8.1 **Embodied energy in MJ/kg of typical construction products**

Building product	Embodied energy (MJ/kg)	Building product	Embodied energy (MJ/kg)
Air-dried sawn hardwood	0.5	Precast tilt-up concrete	1.9
Kiln-dried sawn hardwood	2.0	Clay bricks	2.5
Kiln-dried sawn softwood	3.4	Cement	5.6
Particleboard	8.0	Glass	12.7
Plywood	10.4	Galvanised steel	38.0
Glued-laminated timber	11.0	Plastics, general	90.0
Laminated veneer lumber	11.0	PVC	80.0
MDF	11.3	Copper	100
Hardboard	24.2	Aluminium	170

From Australian Government, 2015. Your Home. Available at www.energyrating.gov.au; Selective extract from Lawson, B., 1996. Building Materials, Energy and the Environment: Towards Ecologically Sustainable Development. Royal Australian Institute of Architects, Red Hill, ACT.

- can be recycled into other wood-based products, and,
- at the end-of-life, if they can no longer be reused or recycled, can either be
    - disposed of in landfill, and if in anaerobic conditions (no oxygen) the carbon in the EWPs can be considered to be retained in storage indefinitely (Ximenes et al., 2013), or
    - used as a biofuel displacing the need to use a nonrenewable fossil fuel and releasing no more CO_2 than was absorbed in the growth of the wood.

8.3.2 Embodied energy and carbon

A major consideration for construction materials is the energy and carbon impact embodied in the materials manufacture. Embodied energy values are often presented in terms of mega-joules per kilogram (MJ/kg) or similar units; Table 8.1 provides some comparisons of process energy requirement (PER)[1] embodied energy values for typical building products including EWPs (Australian Government, 2015; Lawson, 1996).

It should be noted that if CO_2 emission reduction is the focus, and the embodied energy values in manufacture are converted to embodied CO_2 with the carbon-sequestering attributes of wood recognised, then the comparative benefits of sawn timber and EWPs become much more obvious.

8.3.3 Life-cycle assessment

LCA[2] is today the most widely accepted method internationally for the evaluation of environmental impacts of materials, products and assemblies of products, such as buildings. LCA provides a formal process of quantifying total environmental impact and accounts for all inputs (materials and energy usage) and all environmental impact outputs. Life-cycle inventory gathering and LCA analysis of different solid and EWPs has been undertaken in most major timber-producing countries around the world. The summary of the results from these studies is generally reported in the form of published environmental product declarations (EPDs)[3] that are then used by LCA professionals and in LCA software to undertake broader LCAs, including LCAs of full buildings. A number of international EPD websites for wood products are listed at the end of this chapter.

As discussed in Chapter 7, there are a number of important considerations with the use of LCA, including establishing functional units, allocation methods, system

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[1] Process energy requirement (PER) is a measure of the energy directly related to the manufacture of a material. It includes the energy consumed in obtaining and processing of the raw materials, transportation between manufacturing processes and the actual manufacturing process. PER is the simplest way to quantify embodied energy. Many material embodied energy figures quoted are based on PER.

[2] Life-cycle assessment determines the environmental impacts of products, processes or services, through production, usage and disposal.

[3] An environmental product declaration (EPD) is a verified document that reports environmental data of products based on life-cycle assessment (LCA).

boundaries and analysis assumptions. Each of these will be examined briefly here in regard to EWPs.

- *Functional unit* is a prime consideration particularly for any type of comparative LCA studies. The common functional unit for panel and solid EWP products (such as glulam, CLT, etc.) is by volume, generally m^3, whilst for I-beam products, a lineal unit is used, that is, 10 lineal meters or 1 km of I-beam.
- *Allocation* is the practice of attributing impact or benefits to a specific part of a process that results in multiple outputs or coproducts, particularly important for wood-based products. Typically a 'mass' allocation method is used with EWPs, unless the economic value difference is at least 10 times greater between products from a multioutput process; then a suitable revenue-based allocation principle should be applied (AWC, 2013).
- *System boundaries* define the limits of what flows are included within an LCA. Cradle-to-gate LCAs for EWPs include all of the stages up to completion of the manufacturing process within their system boundary. For example, the production of a hardwood glulam beam total cradle-to-gate LCA would involve the hardwood forest, sawn timber processing, and the glulam manufacturing segments and would include all of the inputs into the system in terms of raw materials and energy and all of the outputs: air and water emissions and solid wastes; see Fig. 8.1.

The LCA study for different materials and products establishes the links between the life-cycle inventory results and potential environmental impacts. Typical environmental impact category indicators reported in EPDs include global warming, ozone depletion, acidification, smog and eutrophication. Once collected, these LCA results can then be used either internally, to assess the impact of the material manufacturing process and target hotspots if necessary for improvement, or externally in marketing and promotion, or for broader LCA studies of complete buildings. As mentioned in the previous chapter, though the whole of building LCA studies throughout the world often include material analysis with slightly different functional units or boundary conditions, in virtually all instances researchers have found that using wood products compared to alternative construction products results in lower environmental burdens.

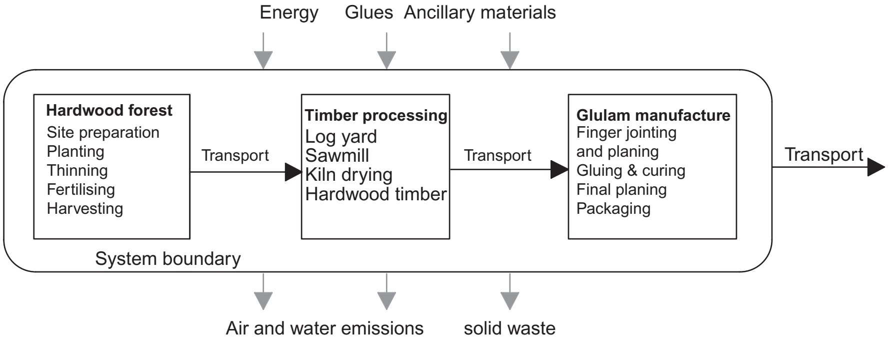

Fig. 8.1 Example of a system boundary for hardwood glued laminated timber manufacture.

8.4 Usability of wood fibre

Recovery of wood fibre from the harvested log is a major consideration for the timber sector. With the logs used to produce plywood, LVL and PSL, it is possible to recover around 50% of the available log fibre volume. This figure is comparable to sawn timber recoveries. Sawn timber waste is largely a consequence of cutting rectangular timber from a log that is approximately round and tapered. Plywood, LVL and PSL involve log peeling but there exists a spindle core that cannot be peeled as indicated in the Agree Asia-Pacific Forestry Commission (2008) report on 'Availability and Use of Mill Residues'. Other EWPs such as particleboard are able to use sawmilling waste. Finally there is the issue of end-of-life disposal. At end of life, EWPs can be broken down and recycled into feedstocks for other composite materials; a range of processes exist to do this though it is noted that this can be expensive and require economies of scale to be successful (Taylor et al., 2005). End-of-life issues are also addressed in Chapter 7.

8.5 Finger jointed timber

8.5.1 Applications

Finger jointed timber (FJT) is a substitute for sawn timber that involves end jointing of short, unmarketable lengths of timber (see Fig. 8.2). Its traditional use is in domestic house construction for wall studs and floor and roof framing members. There are some reservations over the reliability of FJT where it forms the sole load path and the loading is tension or bending action, but this is much less of an issue in domestic house construction where the level of structural redundancy is quite high. If suitable manufacturing safeguards are put in place, such as proof loading of finger joints, FJT can be a highly reliable form of timber construction. Care needs to be taken using FJT in the bottom chords of roof trusses which are loaded in tension under dead and live load, especially in girder trusses, which may be the sole load-carrying element for quite large roof areas.

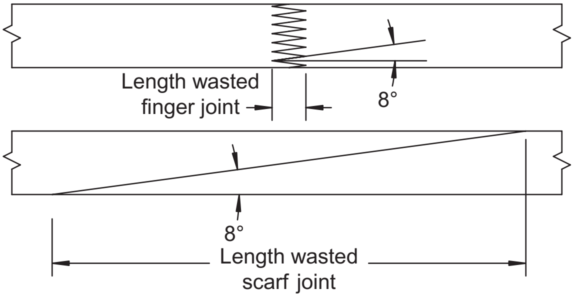

Fig. 8.2 Finger and scarf joining timber illustrating the efficiency of finger jointing.

8.5.2 Manufacture

The short lengths arise from normal sawmilling operations. Because tree trunks taper, sawing methods result invariably in the production of short lengths. These tapered sections can be further processed into chip and used to make other products as discussed in the following, but the short nontapered sections can be end joined to form longer, usable lengths. Finger profiles are cut into the ends, adhesive is spread on one or both mating faces and the fingers pressed together. The concept of finger jointing evolved from scarf jointing, a much more wasteful jointing strategy, given that the taper angle is usually around 8° (see Fig. 8.2). Scarf jointing also possesses processing disadvantages. With scarf joints, external clamping is required while the adhesive hardens, but with finger joints, the fit is sufficiently close that the timber, with care, can be handled prior to glue hardening. Production lines are usually arranged such that the joints are heat- or radio-frequency cured and approach their final strengths within a short time frame.

8.6 Structural glued laminated timber (glulam)

8.6.1 Applications and description

Structural glulam is an adhesively bonded timber product that is used extensively in larger scale timber structures (eg, Fig. 8.3), but also in domestic construction. In its most general form, it comprises an assembly of laminates having thicknesses in the range of 20–50 mm. There is no upper limit to the width and depth and members up to 1500 mm or more in depth, and 250 mm or more in width can be manufactured and used to freely span substantial distances where used in an appropriate structural form. It can be curved, pre-cambered and is thus capable of producing a wide range of architectural shapes. Some structural forms involve bonded reinforcement of high-strength fibres (carbon, glass, aramid) or embedded steel; other forms involve curved shapes that permit external tension reinforcement such as with tied arches.

Fig. 8.3 Glulam beams (manufactured by APA Member American Laminators) create the arched curve profile of the roof (designed and erected by Western Wood Structures) of the Lemay Car Museum under construction in Tacoma, Washington, USA.
Photograph courtesy APA, Tacoma, USA.

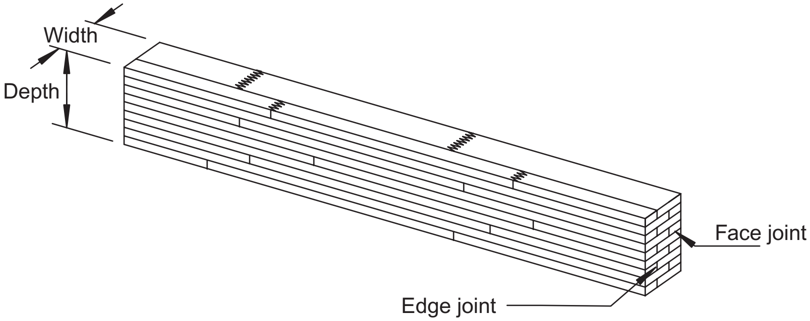

Fig. 8.4 Large glulam member, with finger joints through the wide face. They can also be placed on the side face of the laminations.

Glulam is the modern replacement for large-size cross-section members that were previously sawn from larger trees. Large section members are now produced from relatively small-diameter sawlogs by laminating and adhesive bonding and have the added advantage that the pieces can be curved. As the large sections can span considerable distances, it does mean that the individual component pieces have to be end (finger) jointed, face jointed and, for wider glulam, also edge jointed; see Fig. 8.4.

Structural glulam has engineering design (bending and tension strengths most importantly) properties exhibiting coefficients of variation of between 15% and 20% which is a consequence of redistributing and reinforcing defects, especially knots and sloping grain. The finger jointing quality is critical to the maintenance of consistent properties.

8.6.2 Manufacture

The major steps involved in the manufacture of glulam are outlined here:

- *Sawing and drying of the laminates*. Glulam manufacture commences with sawing logs to typical sawn timber (lumber in Canada and the USA) sizes and then kiln drying the material to a moisture content of around 10%. Typical laminating lengths are in the range of 2–4 m.
- *Grading*. The laminates are graded with the higher grade material being placed in the outer zones where higher stresses are present in bending applications. It is also possible to use higher modulus of elasticity (MOE) material, which has a relatively high level of defect provided these defects are removed by docking.
- *Finger jointing*. Laminating lengths are finger jointed to form laminates of length equal to the length of beam that is to be manufactured after which the end joint adhesive is allowed to cure.
- *Face dressing and initial assembly*. Laminates are commonly dressed, then assembled dry in the sequence for which they are required in the final assembly.
- *Glue spread, lay-up and pressing*. Laminates are passed through a glue spreader that coats the face and edge joints, which is followed by assembly in a press. Once glue spread and assembly are complete, face pressures are applied of approximately 600 kPa for softwoods and 1 MPa for hardwoods. Any curvature required for a particular design is imparted to the beam

within the press. It is also common for pre-camber to be built into the member to counteract the effect of dead loading, but care is then required in construction to ensure that any beams are installed with the camber upwards.

- *Adhesive cure*. Adhesive is allowed to cure usually by heating the pressed assembly.
- *Finishing*. The assembly is completed by dressing to size, sanding and the filling of minor defects. It is usually wrapped in plastic for delivery to a construction site. Care is required to keep the product dry once it arrives on site.

8.7 Structural composite lumber

8.7.1 Applications and description

Structural composite lumber (SCL) is of US origin as the use of the term *lumber* suggests. The American Forest & Paper Association's (AFPA, 2008) 'Structural Composite Lumber & Glued Laminated Timber Awareness Guide' (undated) categorises SCL according to its method of manufacture. The first manufacturing method relies on rotary peeling of logs (laminated veneer lumber or LVL and parallel strand lumber or PSL), and the second (laminated strand lumber LSL and oriented strand lumber OSL) relies on a stranding process; see Fig. 8.5 illustrating the physical makeup of such products.

SCL products are generally used as beams and can substitute for sawn timber in application; that is, they can be used as joists, rafters and similar structural elements. They have engineering design properties with a very low statistical coefficient of variation of between 10% and 15%, which implies that, on average, the wood fibre is being used more efficiently. According to the AFPA, "Advancements in technology have given SCL manufacturers the ability to take apart a smaller log, sort the pieces, apply adhesive, and reassemble them back together into an engineered product. SCL products have grown in popularity because of the ability to manufacture long lengths and large cross-sectional dimensions with consistency".

Fig. 8.5 Laminated strand lumber (*top*) and laminated veneer lumber (*bottom*).

8.7.2 Manufacture

Again, following the AFPA, “structural composite lumber products are produced through two primary log-processing methods - stranding and rotary peeling. The manufacturing process for all SCL typically includes sorting and aligning the strands or veneer, applying adhesive, and pressing the material together under heat and pressure. By redistributing natural defects, sorting for stiffness or density, and through quality control procedures, the resulting product is uniform. Stranding slices the entire log into 3-inch to 12-inch strands, similar to grating a block of cheese”. Further detail is available on the AFPA website.

8.8 Structural I-beams

8.8.1 Applications and description

According to Leichti et al. (1990), structural timber I-beams are old in concept, having been used in aircraft construction in the 1920s, but have only been widely used in housing more recently. They have found their way into heavier timber construction as intermediate as opposed to main frame members, where they are used as floor and roof joists or purlins. They take the use of the wood fibre to an extreme of efficiency. They are very easy to handle due to their lightweight nature and perform extremely well structurally due to their high strength-to-weight ratio. Typically they use webs of OSB as shown in Fig. 8.9 or plywood (see Fig. 8.6), and flanges of sawn timber or LVL.

8.8.2 Manufacture

The flanges are either sawn timber or SCL, the production of which has been described earlier. With sawn timber flanges it is usually necessary to finger joint the component pieces, which can then be proof tested in tension prior to forming the final

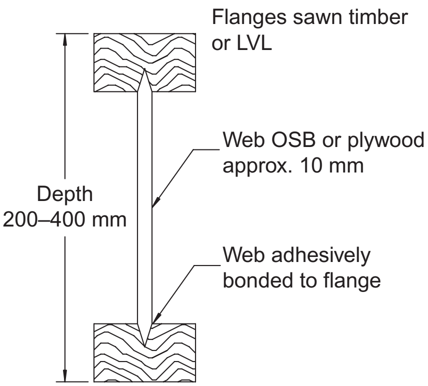

Fig. 8.6 Typical timber I-beam cross-section.

product. Subsequently, the flanges are grooved, the taper cut into the OSB or plywood web, the webs scarf jointed and finally the flanges and webs bonded. All adhesive curing is undertaken using radio frequency methods.

8.9 Cross-laminated timber

8.9.1 Application and description

CLT is the most recent of the EWPs to enter the marketplace and to find widespread use. The initial developments took place in Austria and Germany and have since become a worldwide phenomenon with rapidly increasing volumes of production. An excellent description of the product is provided by Brandner (2013), who describes it as follows.

> "Cross laminated timber (CLT) constitutes a plate-like engineered timber product which is optimised for bearing loads in and out of plane. CLT is composed of an uneven number of layers (in general terms three, five, seven or even more), each consisting of side-by-side placed boards (or beams), which are cross-wise arranged to each other normally under an angle of 90 degrees and quasi rigidly connected by adhesive bonding." See Figs. 8.7 and 8.8 for typical details.

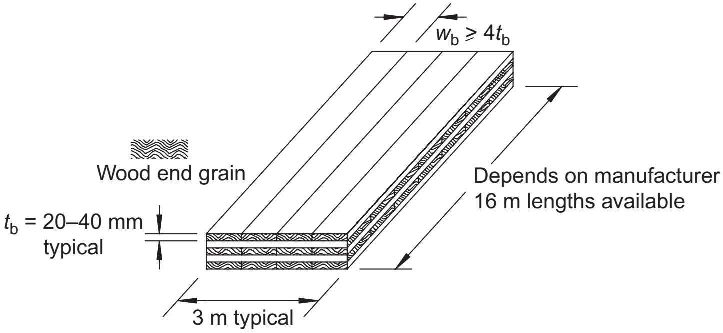

Fig. 8.7 Diagrammatic lay-up details of five-layer CLT.

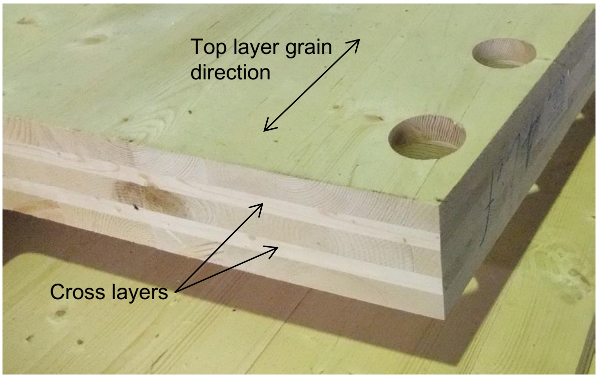

Fig. 8.8 CLT with a typical five-layer layout. Three- and seven-layer layouts are also available.

The panels are used in single- and multistorey construction and have the great advantage that the panels arrive on site ready for assembly and with cut-outs included. As will be seen later, this product is being promoted as somewhat revolutionary in that it lends itself to prefabrication similar to tilt slab construction with concrete but with considerable advantages over that medium.

8.9.2 Manufacture

The layers are in the range of 12–45 mm thick with a typical width of four times the thickness, and fabricated from finger jointed mill shorts and full-length material. The panels are then spread with adhesive full width, the assembly is pressed with either hydraulic or vacuum bag equipment and then allowed to cure. For large-scale production, polyurethane adhesives are preferred as they are more easily cured under ambient conditions. Full details of the manufacturing process are provided by Brandner (2013).

8.10 Oriented strand board

8.10.1 Applications and description

OSB is a product of US origin described by Elmendorf (1949) and later patented (Elmendorf, 1965). According to Bailey (1996), a product called waferboard appeared on the market in the 1970s, and the modern product now known as OSB developed from there. OSB finds application in a variety of board applications including shear wall, as sheathing, flooring and is used extensively as the web for I-beams (see Fig. 8.9).

8.10.2 Manufacture

OSB can be manufactured from low-grade wood plantation species which is chipped into flakes typically 0.5–0.7 mm thick, 19–38 mm wide and around

Fig. 8.9 Oriented strand board flooring supported on I-beams with OSB webs. Photograph courtesy APA, Tacoma, USA.

76 mm long; the long direction corresponds to the grain direction. Chip size is chosen to optimise resin use; usually phenol formaldehyde. The boards (typically 2440 × 1200 mm) are usually made up in three layers with the chips in the middle layer aligned approximately parallel to the 1200-mm edge and approximately parallel to the 2400-mm edge in the two surface layers. This gives the board usable strength and stiffness in the two directions. Typical lengthwise and transverse modulus of rupture (MOR) values are 30 and 20 MPa and MOE values 5 and 2 GPa. OSB finds application wherever particleboard and plywood are used (flooring, sheathing, but is considered unsuitable for concrete formwork where plywood is preferred) and, increasingly, in more highly transformed timber products such as structural I-beam webs. The major steps involved in its manufacture are described briefly in the following.

- *Debarking and log docking*: After harvesting, transport and assembly at the mill site, the logs are debarked then docked into billets of a length that can be accommodated by the chipper or waferiser.
- *Chipping or waferising*: The billets are reduced to chips or wafers (sometimes called flakes or strands) in a machine that contains several hundred serrated knives and which rotates at several hundred revolutions per minute. The fines are separated and then added back to the surface layers to provide a better quality finish.
- *Drying*: The wafers are next dried to approximately 5% moisture.
- *Resin application*: The wafers are then mixed with a powdered resin or are sprayed with a liquid resin at a rate in the range of 2–6%.
- *Lay-up*: Lay-up occurs on a running belt which aligns the wafers by means of electrical field or by vibration. The surface layers are aligned in the belt direction and the core transversely. The unpressed lay-up is between 100 and 200 mm high.
- *Pressing*: The mat is cold pressed to allow air and water to escape and then hot pressed at around 205°C for approximately 10 min. The sheets are allowed to cool then finally finished to size.

8.11 Plywood

8.11.1 Applications and description

Plywood is a sheet product used extensively in engineering construction in applications such as formwork for concrete, flooring, sound barriers in freeway construction, composite beams (webs in nailed box beams) as well as in nonstructural applications such as furniture, wall panelling and the like. It is manufactured from larger diameter logs by a peeling process and performs its job economically by producing a high-strength product with very consistent properties; that is, the engineering design properties have a low statistical variability of approximately 15%. The veneers are typically of the order of 1.5–3 mm thick and are arranged such that the wood grain runs at 90° in alternate plies, which imparts usable strength and stiffness in both sheet directions. It is typically built up with an odd number of plies to thicknesses of between 3 and 30 mm.

8.11.2 Manufacture

The following is a brief outline of the manufacturing process.

- *Conditioning*: Following harvesting, transport and assembly at the mill site the logs are first conditioned in a heated water bath or by steam curing. It is important that good-quality logs be used and that these not be allowed to dry and split as this would severely disrupt the veneer-peeling process that follows.
- *Peeling*: Logs are cut into billets and located in the spindles of the lathe. Because the logs are not perfectly round and taper to some extent, this is best achieved using a laser scan which establishes how to best position the billet to achieve maximum log recovery. The veneer is then peeled in its wet condition by rotating the billet against a lathe knife to form a continuous ribbon. Any splits in the log would disrupt the continuous ribbon, which is a major reason that care must be taken with the logs in the stacking area by continuously spraying them with water.
- *Drying*: The ribbon can be clipped into shorter lengths before drying or dried as a continuous ribbon. Drying is continued until the veneer moisture content is between 6% and 12%.
- *Grading*: The veneers are next graded according to defect level. Typically, in appearance grade plywood the higher quality defect-free veneers would be used for the outer plies.
- *Lay-up and bonding*: Adhesive is applied to the veneers after which they are ‘laid up’ with the grain at right angles in alternate veneers. Only one face of each veneer is glued. The sheets are typically rectangular in shape. The grain direction in the two face veneers is aligned in the long sheet direction.
- *Pressing*: A cold press is applied to facilitate adhesive spread. After cold pressing, the plywood is hot pressed between platens heated to a temperature of around 140 °C and a pressure of around 1 MPa for a period of approximately 10 min.
- *Finishing*: After pressing the sheets are sprayed with water allowed to cool after which they are trimmed to precise dimensions and the faces are then sanded.

8.12 Chipboard or particleboard

8.12.1 Application and description

Chipboard (US, European and UK terminology) or particleboard (Australian and New Zealand terminology) involves the adhesive bonding of wood chip into a board product. Because of its low cost it is used extensively as a structural product in flooring in Australia and New Zealand. It has even been used as a shear wall in domestic housing. Elsewhere, the product is used only in furniture construction and nonstructural applications.

Chipboard has the virtue that it can be manufactured from relatively low-cost wood chip. Typically it has a MOE in the range of 2–3 GPa and a MOR in the range of 9–20 MPa. Its MOR parallel and perpendicular to the long edge of the sheets does not differ greatly although there is a slight tendency for it to be stronger in the long-edge direction. Its density is usually in the range 250–400 kg/m^3.

8.12.2 Manufacture

Wood particles or flakes comprise 80–90% of the weight of particleboard, usually made from softwood species that come either from forest thinnings or sawmill

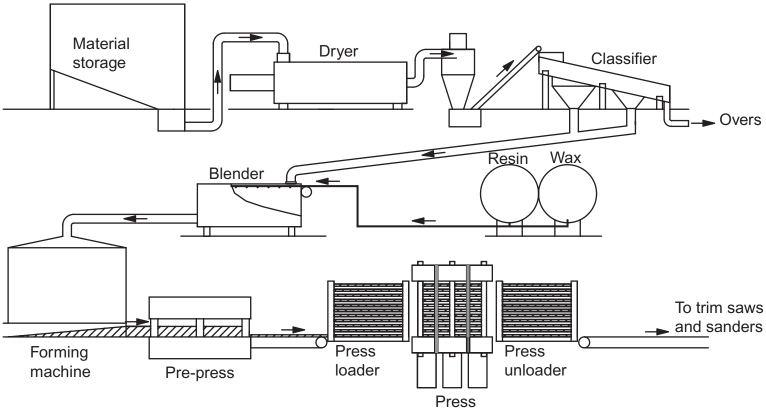

Fig. 8.10 Chipboard/particleboard manufacturing process illustrated.

residues. The binder (resin or adhesive) and the amount used play a key role in the stability of the final board. Thermosetting synthetic resins are generally used which, because their formulation can be varied, have the advantage of flexibility of curing time.

The amount of resin used is usually in the range 4–12% of dry wood; however, this proportion may vary according to the type and size of wood fibres or particles. For example, in three-layer particleboard, the coarser core material may contain 4–10% resin, while the finer surface layers may have 10–12% resin.

Heat energy is required for drying, hot oil for the hot press and steam for pressing operations. Wood particles are passed through driers that reduce their moisture content to 3–5%. Most modern driers are direct fired in that the particles are dried by direct contact with hot gas from the burners.

Resin, in the form of a liquid, is forced through nozzles and sprayed onto the particles, in a separate 'blender', after which it is formed into a mat which is then subjected to heat and pressure to cure the resin and produce a board of the required thickness. Boards are sanded prior to sale or prior to prefinishing with various surface and edge treatments. The rough panels of particleboard are trimmed after pressing and can be cut to size as required. The overall process is illustrated symbolically in Fig. 8.10.

8.13 Fibreboard

8.13.1 Fibreboard types and applications

The primary types of structural fibreboard include MDF and hardboard, also known as high-density fibreboard or masonite. The latter name is strictly a trade name. Only the application and manufacture of medium fibreboard is described herein.

MDF is a wood-based composite board that has a higher density than particleboard, in the range 600–800 kg/m^3, an MOR of 40 MPa and a MOE of 3 GPa. Because it is easier to machine and has good weathering characteristics, it is tending to replace particleboard in applications such as furniture, cabinet making, joinery, craft work and flooring. It can be made from a variety of fibrous products including softwoods, hardwoods, bagasse, rubber wood, cotton stalks and other raw products.

8.13.2 Manufacture

MDF comprises fibrous materials that have been broken down into the tracheids and vessels that make up the cellular structure of wood (repeated in the following). The basic procedure is detailed here:

- *Debarking and log docking*: After harvesting, transport and assembly at the mill site, the logs are debarked then docked into billets of a length that can be accommodated by the chipper.
- *Chipping*: The billets are reduced to chips.
- *Pulping*: Pulping is the process that most distinguishes MDF from particleboard. In this process the chips are broken down into the tracheids and vessels that make up the basic cellular structure of wood (as described earlier). The end result of this process is a dispersed fibrous mass.
- *Blowline*: Following defibration the fibres enter the blowline where it is sprayed with wax and resin.
- *Lay-up*: Lay-up occurs on a running belt followed by a cold press that produces a mat of thickness between 250 and 600 mm.
- *Pressing*: The mat is hot pressed during which time the adhesive cures. The sheets are allowed to cool then finished to size. Various finishes may be added.

8.14 Adhesives

8.14.1 Service conditions

The whole topic of EWPs cannot be left without dealing with the issue of wood adhesives. Those used in EWPs are classified as suitable for use in one of three conditions; see ISO 20152.1.

- Service Class 3 (SC3)—Full weather exposure
- Service Class 2 (SC2)—Covered weather exposure
- Service Class 1 (SC1)—Indoor exposure where wood moisture cannot rise above 12% except for short periods such as a few weeks per year.

8.14.2 Adhesive types

Wood adhesive science and technology is a highly specialised field; no attempt is made to cover the technology herein. Modern industrial adhesives are based largely on oil by-products whereas vegetable and animal glues were used historically. The principal types in use include the following.

- Urea-formaldehyde (UF) has the unfortunate property of degrading in hot and moist conditions so that it is only suitable for use in SC1 conditions. It releases formaldehyde, a carcinogen, in the process as do all formaldehyde adhesives although at levels well below those that are harmful to humans.

- Melamine urea formaldehyde (MUF) is a higher performing adhesive suitable for use in SC2 conditions although there is not universal agreement on that point as some MUFs will meet the requirements of SC3 of some national standards.
- Phenol formaldehyde (PF), resorcinol formaldehyde (RF) and phenol resorcinol formaldehyde (PRF) adhesives are regarded as providing the benchmark for durability and so are recommended for SC3 without exception.
- Polyurethane (PUR) glues are becoming more popular and are used extensively in the manufacture of CLT. Under dry conditions they tend to be slightly stronger than PRFs but have the characteristic that they lose strength in water-saturating tests but recover that strength when they dry. They also meet SC3 conditions of most national standards.
- Epoxy, usually a two-part mix system, cures under a wider range of temperatures and moisture content than do other glues, does not require pressure while curing, and has good gap-filling properties. They are not used in the manufacture of EWPs but are employed in some joint designs, specifically for the grouting of steel rods when forming the knee and ridge joints of portal frames.

8.14.3 PURs and wet bonding

The issue of PURs and poor wet bonding characteristics has received recent attention (Klauser, 2014). One characteristic that they have is in presenting a barrier to moisture penetration of wood which is not the case with the formaldehyde adhesives, and this may well be the explanation for their behaviour. Because of the impact of CLT on building construction, it is probably prudent to protect it from the weather.

8.15 Future trends

8.15.1 CLT construction

CLT is the most recent EWP to impact the market and increasing volumes are coming into use in markets that have previously been the preserve of steel and concrete, namely, inner- and outer-city multistorey construction. This trend is exemplified in the following case studies.

8.15.2 Forté Living, Melbourne, Victoria, Australia

Forté Living is a 10-storey apartment building in Melbourne, Victoria, Australia. The outer façade of the building is shown in Fig. 8.11 and the basic structural components can be seen during the construction phase in Fig. 8.12. It is constructed predominately of CLT and at 32.2 m in height it is the world's tallest modern timber apartment building (Lend Lease, 2013). The building consists of a reinforced concrete structure at the ground floor level for retail use (this podium also protects against termite attack) and nine stories above housing 27 apartments all constructed from CLT (floor panels, load-bearing walls, lift shafts and fire escape stair) and protected from the elements by an AluBond external rain screen.

Fig. 8.11 Forté building, Melbourne, Australia.
Courtesy Forest & Wood Products Association.

Fig. 8.12 Forté Building under construction. All walls, floors/ceilings are constructed from CLT. Note also the small number of workers on site and minimal cranage required.

The building is constructed on a site near Melbourne's waterfront with very poor silty soil conditions. All modern buildings in this area sit on piles, some up to 40 m deep down to bedrock. The use of timber construction being around a fifth of the weight of concrete delivered significant savings in materials, time and cost to the builder in terms of the footing requirements and installation.

The CLT component of the building utilises 759 CLT panels of European spruce, which was grown, harvested and manufactured in Austria and then shipped to Australia in 25 shipping containers. The adhesive used in the CLT product is a formaldehyde-free polyurethane adhesive (PUR), according to EN 301 accredited for: load-bearing and non-load-bearing components for indoor and outdoor application.

Compared to conventional concrete systems, construction of the CLT building was very quick, easy and highly neighbour-friendly. The CLT panels after arriving from Austria were stored in a flat-pack form in a nearby warehouse. The panels required for that day's erection were then trucked to site, effectively only one truck-movement a day (90% less than for an equivalent concrete building). Panels were lifted into place using a small remote-controlled crane and then simply screwed together; the project overall used 5500 angle brackets and 34,550 screws (Nieland and Lend Lease, 2013). The overall build time for the CLT components was estimated by the builder to be 30% faster than concrete. Other benefits to the builder included: reduced preliminary and supervision costs; savings in the time and costs for follow-on trades; reduced holding costs; high precision and quality; low requirements on site labour, only five full-time employees were used on the job; and reduced occupational health safety hazards, no lost time was experienced due to OH&S issues over the build period.

From an environmentally sustainable building perspective, the project had many benefits. First, in regard to carbon storage, 485 tonnes of CLT was used in the building structure; this equates to 216 tonnes of stored carbon or 792 tonnes of CO_2 sequestered in the wood. In comparison to a standard concrete and steel building with higher embodied CO_2 emissions, Forté reduces overall CO_2 emissions by over 1450 tonnes (WoodSolutions, 2015).

A full LCA was conducted by staff at the Centre for Design at RMIT University (Durlinger et al., 2013) which compared Forté with a standard apartment reference building constructed with reinforced concrete. The agreed functional unit for the study was

> *The provision of comfortable living space for its inhabitants and space for commercial enterprise for the duration of 50 years, in a nine story building with 197 m² retail space and 23 apartments with an area of 1558 m² located on Bourke street, Docklands, Victoria*

This study provided the following findings; see Fig. 8.13.

- The Forté building had a lower environmental impact on all assessed categories, compared to the reference building, except renewable energy demand (driven by the renewable energy embodied in the wood in the CLT panels).
- If carbon sequestration[4] is included the Forté building's impacts are 22% lower on global warming potential, if sequestration is not included in this indicator, Forté's impacts are 13% lower.

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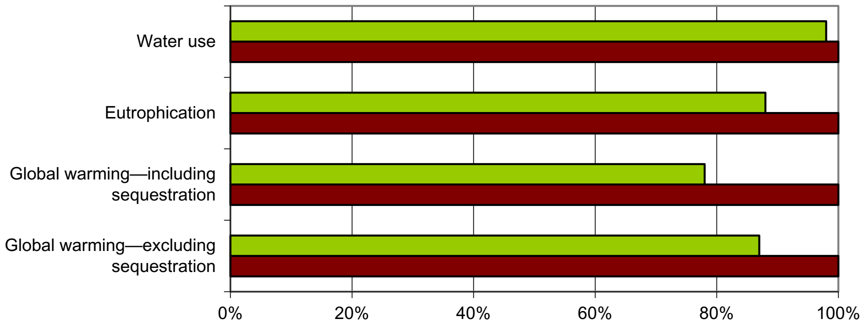
[4] Carbon sequestration is the process of capture and long-term storage of atmospheric carbon dioxide (CO_2).



	Global warming—excluding sequestration	Global warming—including sequestration	Eutrophication	Water use
🟩 Forté building	87%	78%	88%	98%
🟥 RC reference building	100%	100%	100%	100%

Fig. 8.13 Comparison of environmental impacts of the Forté building, predominately constructed of CLT, compared to the reinforced concrete reference building.

- Eutrophication[5] potential is 12% lower and water use 2% lower for Forté when compared to the reference building.
- Even though the CLT is imported (whereas the reference building's materials are mostly locally produced), the Forté building still has a lower impact for materials and transport combined when compared to the concrete reference building.

Using CLT also offers better thermal performance for the building envelope and requires less energy to heat and cool than a concrete building; these reduced energy and water costs the builder advises equating to an average savings of around $300 per year.

All the timber used in the Forté project was certified to internationally recognised sustainable forest management and chain of custody standards. For the CLT from Austria, this being to PEFC (Program for the Endorsement of Forest Certification) standards and for all the Australian timbers to the Australian Forestry Standards (AFS).

The Forté building has won a wide range of awards, and its outstanding environmental performance has been recognised by the Green Building Council of Australia, achieving 'Australian Excellence' 5 Star Green Star – Multi Unit Residential As Built v1 certification.

8.15.3 Other CLT buildings

A recent report in the *International Business Times* (2015) under the headline 'How Wood High-Rises Could Save the Planet' contains the following quote and makes the case for more extensive use of CLT.

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[5] Eutrophication is a syndrome of ecosystem responses to human activities that fertilize water bodies with nitrogen (N) and phosphorus (P), often leading to changes in animal and plant populations and degradation of water and habitat quality.

> *From England and Sweden to Canada and Australia, a new breed of high-rises has started to pop up on skylines and win praise from architects, developers and environmentalists in the era of climate change.*
> *There's the iconic nine-story Murray Grove in London, built in 2009; the upscale Forté apartment complex built in 2012 in Melbourne's revitalized Docklands precinct; a recently completed 95-foot design center tower in Prince George, British Columbia; and a dozen or so others in this emerging genre. And plans are in development for a 30-story residential and commercial tower in Vancouver and a 34-story skyscraper in Stockholm.*
> *They're all made of an advanced building material that's a natural for the age of sustainability and is catching on with governments and developers seeking an alternative to steel-and-concrete buildings, which contribute up to one-third of the world's greenhouse gases: wood?.*

The impact of CLT on construction is clearly a worldwide phenomenon that makes it likely that more buildings will be constructed using that medium in the future, especially in inner-city and outer-city areas. From a manufacturing and sustainability perspective the major interest with CLT in the future will be the wider use of a lower-grade wood resource combined with higher structural grade timber to engineer the best product for application and to maximise the value from the overall forest resource. The future for engineered wood products is very exciting.

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Wood Environmental Product Declaration (EPD) Sites

American Wood Council: http://www.awc.org/greenbuilding/epd.php.
Australia: http://www.epd-australasia.com/.
Canada & US Athena: http://www.athenasmi.org/resources/publications/.
Canadian Wood Council: http://cwc.ca/green/life-cycle-and-epds/.
IBU Germany: http://construction-environment.com/hp4113/Timber.htm.
UK: http://woodforgood.com/lifecycle-database/.

Sustainability of aggregates in construction

9

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9.1 Introduction

Natural aggregate consists of manufactured crushed stone and sand created by crushing bedrock, or naturally occurring unconsolidated sand and gravel. It is a major component of asphalt and concrete and is required in streets, highways, railroads, bridges, buildings, sidewalks, sewers, power plants, and dams—just about every part of the built environment. Aggregate is the world's number-one nonfuel mineral commodity in terms of both volume and value (Fig. 9.1). During 1998, worldwide, about 20 billion tons of aggregate worth about 120 billion Euros were produced (Wellmer and Becker-Platen, 2002). Worldwide demand is estimated to be rising by 4.7% annually (Bleischwitz and Bahn-Walkowiak, 2006).

This chapter describes the natural aggregate industry and methods to sustain aggregate resources. Sections 9.2–9.6 describe aggregates and the aggregate industry, including: Section 9.2—production, transport, reclamation, potential environmental impacts, and methods to manage those impacts; Section 9.3—substitutes; Section 9.4—recycling; Section 9.5—performance in use; and Section 9.6—waste products from aggregates. Sections 9.7–9.9 describe sustainable aggregate resource management (SARM), including: Section 9.7—the environmental, economic, and social aspects of SARM; Section 9.8—the status of SARM; and Section 9.9—general approaches to SARM. Section 9.10 contains four case studies. Section 9.11 discusses the future of SARM and Section 9.12 describes sources of further information.

9.2 Production of aggregate

If aggregate is to be produced from new sources, certain conditions must be met.

- Sand, gravel, or rock must exist in sufficient quantity and quality to make mining worthwhile, and it must be accessible to transportation systems and to markets.
- The property must be of sufficient size to locate a pit or quarry and processing equipment, and be owned by a person or people willing to sell or lease it at a reasonable price.
- The deposit must physically be able to be mined without causing unacceptable impacts to the environment.
- The extraction and processing site must qualify for all necessary permits.
- The approving officials and the public must be convinced that the operation can take place without adversely affecting the environment or their lifestyle. In other words, the operator must be able to obtain a "social license" to mine.

Sustainability of Construction Materials. http://dx.doi.org/10.1016/B978-0-08-100370-1.00009-3

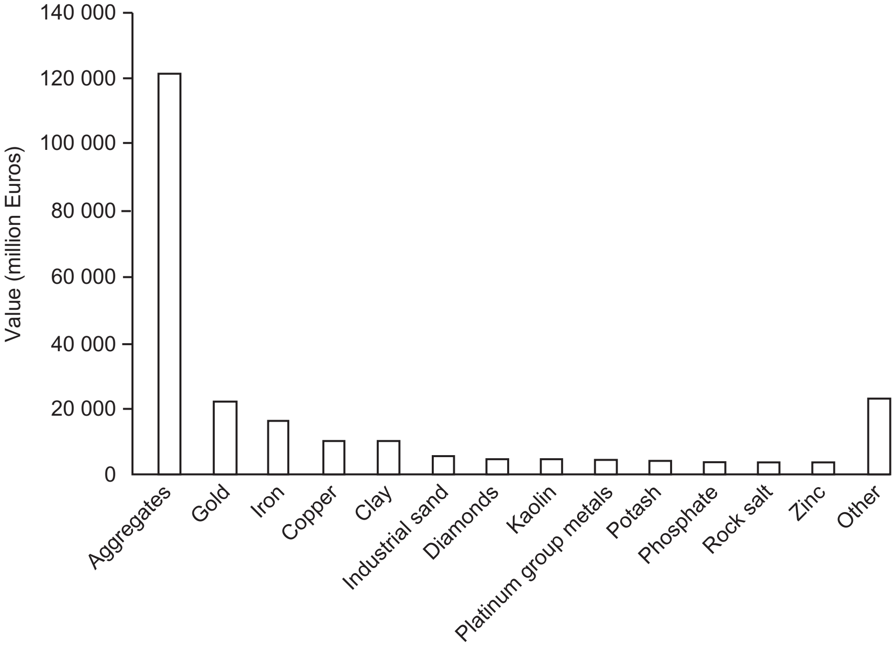

Fig. 9.1 Graph showing the value of worldwide nonfuel mineral production during 1998. Data from Wellmer, F.W., Becker-Platen, J.D., 2002. Sustainable development and exploitation of mineral and energy resources: a review. Int. J. Earth Sci. 91, 723–745.

- The operation must be profitable considering all costs including: exploration, acquisition, permitting, operation, environmental controls, compliance with regulations, transport to market, and reclamation.

The production of aggregate involves extraction and processing of the raw material into a usable product, transport of that commodity to the point of use, and the reclamation of mined-out pits or quarries. The following is a general description of the production of natural aggregate. More detailed discussions can be found in the sources of further information listed in Section 9.12.1.

9.2.1 Extraction and processing

Sand and gravel deposits commonly are excavated from pits utilizing conventional earth-moving equipment. Mining crushed stone generally requires drilling and blasting of solid bedrock (also referred to as ledge or ledge-rock), which breaks the rock into rubble of a size suitable for crushing. Crushed stone and sand and gravel commonly are obtained from dry pits or quarries, but in some settings may be mined from water-filled excavations using dredges mounted on barges, or with draglines.

Sand and gravel or rock rubble at the mine face are transported by truck or conveyor to a processing plant. The material is crushed, passed over a screening device, and sorted according to size (Fig. 9.2). The crushing, screening, and sorting process is

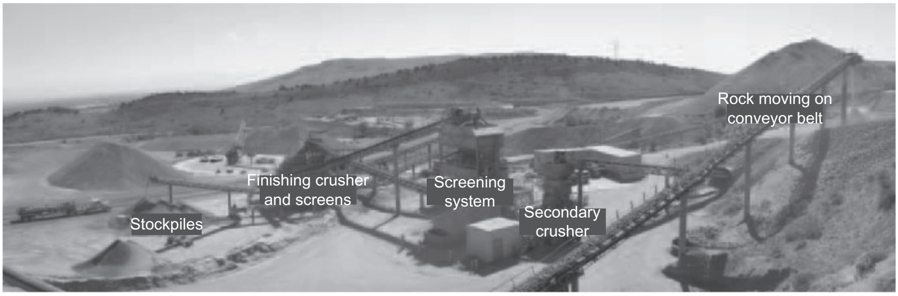

Fig. 9.2 Typical crushed stone processing plant.

repeated until the proper mix of particle sizes is reached. Sand and gravel may or may not be crushed, depending on the size of the largest gravel particles and the desired product. Depending on the specifications of the final product, the processed material may be washed to remove dust. Sand may be screened from the mixture and processed separately. After screening, sorting, and washing (if necessary), the sand and different size gravel/rock particles are moved by conveyors to separate stockpiles where they are stored until sold and shipped.

9.2.2 Transportation

Most aggregate is sold in bulk. Upon sale, aggregate is loaded on trucks, railcars, barges, or freighters for transport to a destination. Aggregate is a high-bulk, low-value commodity, and transportation can add substantially to the cost at the point of use. For example, the cost of transportation of aggregates in the European Union is about 13% of the total cost of the aggregate (Bleischwitz and Bahn-Walkowiak, 2006).

The method of transport depends on a number of factors including volumes of material, distance to the point of use, delivery schedules, and access to rail or water transport systems. Trucks are by far the most flexible and most common means of transporting aggregate. They can be loaded and unloaded at many locations using a variety of techniques and can accommodate most delivery schedules. Rail and barge are much less flexible because they utilize fixed route systems following strict schedules and require considerable investment capital in terms of loading facilities, off-loading facilities, and distribution yards. Trains and barges achieve economy by moving large volumes of aggregate long distances on regular schedules (Hayes, 1991).

9.2.3 Reclamation

Reclamation may be implemented following four reclamation strategies: progressive, segmental, interim, or postmining (Norman and Lingley, 1992). Progressive reclamation immediately follows the removal of aggregate, but may be impractical for operations that must blend mined material from different parts of the pit or quarry. Segmental reclamation follows the removal of minerals in designated sections of the mine, is cost efficient, establishes final slopes as part of the mining operation, and works best in homogeneous deposits. Interim reclamation temporarily stabilizes disturbed areas with fast-growing grasses or legumes, and at a later time implements the final reclamation plan. Postmining reclamation does not begin until the entire mine has been exhausted, which may lead to deterioration of stockpiled soils, a longer revegetation time frame, and high bonding liability (Norman and Lingley, 1992).

The following examples (from Arbogast et al. (2000), unless otherwise noted) illustrate the many different ways that sites can be reclaimed. Reclamation can produce economic benefits by reusing pits or quarries as residential property, industrial and commercial properties, office parks, landfills, golf courses, recreational areas, and botanical gardens. Water-filled pits or quarries are especially well suited for lake-form residential properties, reconstructed wetlands, and water storage reservoirs. These types of reclamation often occur in or near urban centers with large populations. For example, beginning in 1904, Buchart Gardens in British Columbia, Canada, reclaimed 50 acres of an exhausted limestone quarry to create a premier botanical garden (Fig. 9.3).

Some reclamation uses an artistic approach where the site is celebrated as a work of beauty and unique experiences. For example, Robert Smithson, a pioneer in the earthworks-as-art movement, created a circular jetty and canal entitled "Broken Circle" from a sand pit and body of water in the Netherlands. The symmetrical landform is about 40 m in diameter and evokes images of the dikes and polders that are the backbone of the Dutch landscape. Another form of art can be illustrated by the festival stage Dalhalla at Rättvik, Sweden. The stage was built in a former limestone quarry created in an amphitheater shape, which was reclaimed to seat about 4000 people. In

Fig. 9.3 Buchart Gardens, a reclaimed limestone quarry. Notice the cement kiln stack in the background.

this unique setting, Dalhalla hosts operas, choir music, jazz and big band concerts, symphonies, and chamber music (Langer, 1999).

Quarry Cove, on the Oregon coast, USA, is a quarry that was converted into a man-made tidal zone nourished by wave action. The site was designed as an educational tool where visitors (the site is wheelchair accessible) can view nature taking its course as marine life invades the area. A mined-out sand and gravel pit along the South Platte River in Littleton, Colorado, USA, was reclaimed as a natural wildlife area. The design made use of native seed mixes and incorporated trails, fishing along the river, and educational tours at a nature center. The South Platte Park is one of the largest wildlife parks within city limits in the USA.

9.2.4 Potential environmental impacts and their mitigation

The overall contribution of aggregate extraction to resource depletion, competing land uses, global warming, and energy use is rather low (Bleischwitz and Bahn-Walkowiak, 2006). Consider, for example, competing land uses. All types of mining and quarrying in the EU-15 during 2003 were estimated to use 0.2% of the land compared with 0.6% for industry, commerce, energy production, and wastewater treatment; 2.0% for transportation infrastructure; 2.3% for residential; and 41.5% for agriculture (EUROSTAT, 2003). The impact is even less when considering aggregate mining alone. For Germany, the land used for the extraction of sand, gravel, and crushed rock was equivalent to less than 0.005% of the total area of Germany (Gwosdz and Röhling, 2003).

Nevertheless, aggregate extraction and processing cause environmental impacts including changes to the landscape, noise, dust, vibrations from blasting, and degradation of groundwater and surface water. Potential environmental impacts and methods for mitigation are briefly discussed in the following. More detailed discussions can be found in the sources of further information listed in Section 9.12.1.

9.2.4.1 Changes to the landscape

The most obvious environmental impact of aggregate extraction is a change to the landscape, generally from undeveloped or agricultural lands to a pit or quarry. Careful site selection can minimize the amount of surface area that must be disturbed by resource extraction. Preproduction site inventories can identify rare or endangered species so that habitat can be set aside, selected species can be relocated, or extraction operations can be suspended during critical breeding or migrating seasons. Loss of habitat can be compensated for by creation or improvement of buffer areas and off-site habitat designed to retain the characteristics of the original habitat. After closure, the pit or quarry may be reclaimed to function as the original habitat. Progressive, segmental, or interim reclamation can speed habitat recovery.

The area of extraction may have contained important archeological, paleontological, or geological features that can be identified during pre-quarry inventories. Ironically, such features may be recognized only after aggregate operations begin because aggregate extraction uncovers a relatively large area at a relatively slow pace, sometimes leading to serendipitous discoveries.

9.2.4.2 Noise and dust

The primary source of noise and dust from aggregate extraction is from vehicle movements, processing equipment, and blasting. Aggregate producers are responsible for ensuring that the noise and dust emitted from the pit or quarry do not exceed regulated levels. Carefully prepared and implemented noise and dust control plans can keep emissions within the required limits. The size and design of blasts can be modified to limit generation of noise and dust. Blasting can be scheduled for certain times of the day and restricted during adverse weather conditions. Low-noise equipment and dust suppression or collection systems can significantly reduce impacts. Equipment that is noisy or generates dust can be located so that naturally vegetated areas, landscaping, earthen berms, quarry walls, stockpiles, and topographic barriers shield or absorb noise and block the wind that transports dust. Equipment that generates noise or dust can be located in sound-deadening, vacuum-equipped enclosures (Fig. 9.4). Proper location and surface treatment of haul roads and careful routing of trucks can help reduce noise and dust. Conveyors can be used instead of trucks for in-pit movement of materials.

9.2.4.3 Vibrations from blasting

Blasting may occur daily or as infrequently as once or twice a year, and usually is restricted to quarry operations. Most of the energy of a quarry blast is expended on breaking the rock. A small amount of energy is released as vibrations that go through and along the surface of the earth. Some energy from a quarry blast escapes into the atmosphere and causes audible noise and subaudible noise referred to as “airblast” or “air concussion.” Airblast is most noticeable within a structure, and frequently is mistaken for ground vibrations. Airblasts are less likely to cause damage to structures than ground vibration because the mechanics of airblast vibrations are different from vibrations that cause ground shaking.

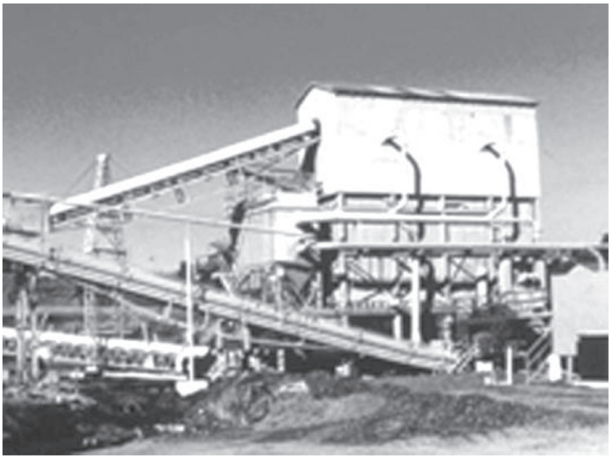

Fig. 9.4 Equipment in sound-deadening, vacuum-equipped enclosure.

Extensive research by the former US Bureau of Mines resulted in ground vibration and airblast standards that are recognized worldwide and have become industry standards for safe blasting (Siskind et al., 1980a,b). Impacts from blasting can be mitigated by maintaining blast vibrations below well-documented limits on ground motion and air concussion (Langer et al., 2004).

9.2.4.4 Impacts on groundwater

The environmental impacts of aggregate operations on groundwater are highly dependent on the local geology, hydrology, and climate. In dry climates, evaporation of water from pits or quarries may lower the water table. In humid climates, precipitation may flow into a quarry and recharge groundwater. Groundwater flow in springs, gaining streams, and wells may be impacted by nearby aggregate operations that pump groundwater from the pit or quarry. Extracting rock from karst areas can have a severe impact on the groundwater, but the impact can commonly be controlled with well-designed and implemented environmental management procedures (Langer, 2001b). Impacts on the water table as a result of dewatering can be monitored by use of observation wells, and recharging aquifers or augmenting flows to streams with water that has drained into the pit or quarry can maintain water levels. In highly permeable deposits, slurry walls or grouting may be necessary to isolate the operation from the water table.

9.2.4.5 Impacts on surface water

Aggregate operations entail removal of vegetation, which can increase runoff. Aggregate extraction may change runoff patterns and promote erosion, which can result in increased sediment in nearby streams. Slope stability, water quality, erosion, and sedimentation are commonly controlled by sound engineering practices. Finished slopes, roads, drainage ditches, and operational areas must fit the particular site conditions. Disturbed areas can be protected with vegetation, mulch, diversions, and drainage ways. Sediment can be retained on site using retention ponds and sediment traps. Regular inspections and maintenance help ensure continued erosion control.

Water from aggregate processing and storm runoff over pit or quarry sites can increase the suspended material (turbidity) in stream runoff. Turbidity is generally greatest at pit or quarry and at wash-plant water discharge points and decreases downstream. Turbidity can be controlled by filtering or by containing runoff or wash water at sediment traps. Aggregate production within stream floodplains may have an impact on stream-channel morphology. Careful hydrologic studies and application of best management practices can allow aggregate to be extracted from certain parts of active stream channels with little environmental impact. However, improper aggregate extraction may cause widespread erosion and loss of riparian habitat (Langer, 2002b).

9.2.4.6 Impacts from transportation

Aggregate is commonly delivered from the pit or quarry to the construction site by truck, which can create problems of noise and exhaust as the trucks pass nearby dwellings. Truck traffic ultimately intermingles with automobile traffic, creating potential hazards such as those caused by trucks that transport other consumer products. The environmental impacts and hazards of trucks can be minimized when the trucks are well maintained and operated, and when automobile drivers allow reasonable space for truck drivers to maneuver and stop safely. Trucks can be equipped with mud flaps and load covers to prevent loose material from being thrown from wheels and loads. Limiting the number of quarry entrances and exits, and constructing acceleration and deceleration lanes at pit or quarry entrances, can allow trucks to enter and exit traffic smoothly. Delivery routes can be designed to minimize interference with neighborhood traffic.

9.2.5 Energy consumption

Producing aggregate requires the use of energy (Fig. 9.5; Moray et al., 2006), which in turn causes the release of greenhouse gases to the atmosphere. The energy consumption required to bring aggregate to a useful state is referred to as *embodied energy.* The energy-consuming activities of aggregate extraction and processing include:

- removing vegetation and soil, building the processing facilities, and otherwise preparing the site for operation;
- drilling, blasting (for crushed stone), and excavating the material;
- transporting material from the excavation site to the processing facility by truck or conveyor;
- processing, including multiple stages of crushing, screening, dust collection, sand classification, washing, and stockpiling;
- load-out and transporting to market.

Embodied energy has been calculated for a number of building materials by a number of investigators. The values vary from one investigator to another because of variations in inputs and analytical approaches. Table 9.1 shows embodied energy values for a number of common building materials, including aggregate. All values are from one investigator (Alcorn, 2001) to ensure conformity. The values in Table 9.1 are generalized. In practice, energy consumption varies greatly from one aggregate operation

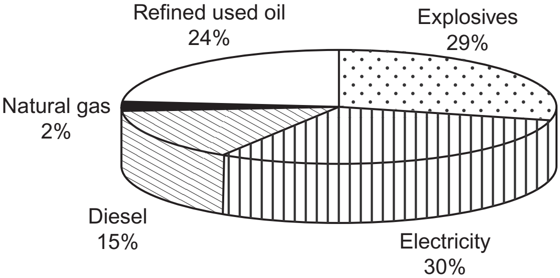

Fig. 9.5 Distribution of energy costs in a typical crushed stone operation.
From Moray, S., Throop, N., Seryak, J., Schmidt, C., Fisher, C., D'Antonio, M., 2006. Energy efficiency in the stone and asphalt industry. In: Industrial Energy Technology Conference, Texas A&M University, College Station, Texas.

Table 9.1 **Embodied energy for some common building materials**

Material	MJ/kg	kW h/ton
Aggregate, general	0.04	11.11
Sand	0.1	27.78
River gravel	0.03	8.33
Crushed stone	0.06	16.67
Asphalt paving	0.2	55.56
Brick, high technology	2.7	750.00
Concrete (40 MPa)	1.5	416.67
Gypsum plasterboard	7.4	2055.56
Timber, pine, gas dried, dressed	9.5	2638.89

From Alcorn, A., 2001. Embodied Energy and CO_2 Coefficients for NZ Building Materials. Victoria University of Wellington Centre for Building Performance Research, Wellington, New Zealand.

to the next, and has been calculated to range from 6 to 139 kWh/ton (0.022–0.5 MJ/kg) (GoodQuarry, 2008). Actual consumption is dependent on a number of factors including: the size of the operation; plant layout and design; the type of rock or sand and gravel being mined and processed; the amount of drilling or blasting required; the type, efficiency, and maintenance of equipment being utilized; the experience and training of drillers, blasters, and other operators; and the method of transport and distance to market.

Methods to reduce the embodied energy in aggregate resources include:

- efficient plant design;
- proper drilling and blasting to create appropriately sized crusher feedstock;
- selecting the right equipment (eg, matching the crusher to the rock being processed, or matching motors to the equipment being used) and operating the equipment properly (eg, monitoring drill rates, or matching the feed rate to the crusher);
- properly maintaining equipment (eg, drilling, crushing, processing, onsite power generation, dust collectors, water pumps, conveyors, excavating equipment, and trucks);
- reducing idle time of trucks, maintaining haul roads, driver education and awareness.

Generally the transportation of finished goods to the customer is not included in the embodied energy calculations for a product. However, aggregate is a high-bulk commodity, and transportation can be a significant part of the put-in-place cost. For comparison, Eastman (1980) reported that the distance one liter of fuel can move 1 ton of material is 23 km by truck, 78 km by train, and 198 km by barge.

9.2.6 Managing impacts through best management practices

Limiting environmental impacts from aggregate mining commonly requires following best management practices, which are available as handbooks and guidelines published by various organizations including government agencies (eg, Ministry of Energy and Mines, 2002) and industry associations (eg, Groundwork, 2001). Industry practices have become so advanced in many industrialized countries that aggregate extraction adhering to best management practices can be considered a temporary, rather than permanent, land use (Wellmer and Becker-Platen, 2002).

Increasingly, aggregate companies are receiving certification from the International Organization for Standardization (ISO) by complying with ISO 14000 (ISO, 2007), a voluntary international standard for environmental management. This standard is primarily concerned with the activities an organization takes to minimize harmful effects on the environment and to continually improve its environmental performance.

9.3 Substitutes and manufactured aggregates

Crushed stone can generally substitute for sand and gravel, and vice versa. There are only a few naturally occurring materials that have the properties necessary to make an in-kind replacement for aggregate. Shells, caliche (carbonate-cemented soil typical of arid areas), and clinker (rock hardened by heat from natural coal fires) can substitute for aggregate in some low-specification applications. Some manufactured products can also substitute for aggregate. Slag, a by-product of iron and steel refining, can sometimes be used as aggregate. Perlite, vermiculite, and some types of clay and shale can be expanded by heating and used as lightweight aggregate (Bush et al., 2006; Hack and Bryan, 2006). Occasionally, other materials have been used as substitutes for aggregate in concrete and asphalt products designed for special applications. Shredded rubber tires are used in asphalt to reduce tire noise. Broken glass or cullet are sometimes used in aggregate base courses. Burned coal cinders are used in cinder blocks (Hack and Bryan, 2006). Recycled brick or concrete block can substitute for low-specification uses. There are some building materials that can replace the function of products made from aggregate. For example, dimension stone, wood, or steel can replace concrete in some applications.

9.4 Extending aggregate availability through recycling

Demolition of roads and buildings generates large quantities of waste. Previously, most waste was disposed of in landfills. Today, asphalt pavement and cement concrete are commonly recycled (Fig. 9.6), thus saving huge amounts of space in landfills and reducing the amount of natural aggregate needed to be mined. The following is a general description of recycling of asphalt and cement concrete.

When aggregate has been used in an unbound form such as railroad ballast, it generally can be scooped up and reused with minimum reprocessing. However, aggregate is commonly used in a bound form as asphalt or cement concrete, which complicates recycling. Concrete commonly contains reinforcing steel bars, which require magnetic separation. Recycled concrete also contains cement paste, which has markedly different properties from aggregate. This affects the overall properties of the recycled material. Consequently, recycled concrete generally is used in low-specification applications, such as base course, and even then it is generally

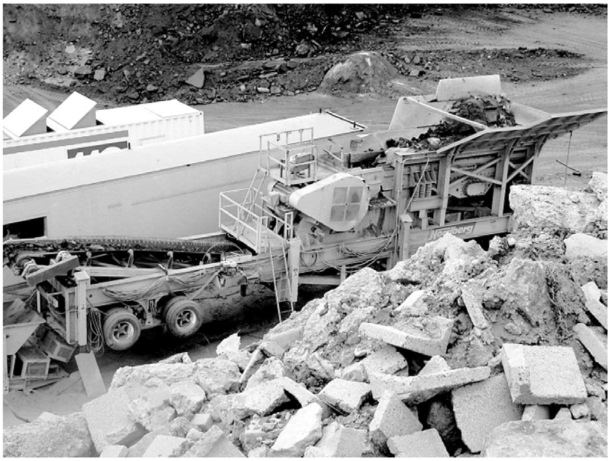

Fig. 9.6 Recycling concrete using a portable crusher.
Photo courtesy of Metso Materials, Tampere, Finland.

blended with fresh aggregate. Recycled concrete is generally not used in new concrete (Hack and Bryan, 2006), although there is potential to use recycled concrete in concrete production.

Asphalt roads and parking lots commonly are recycled. Worn-out surfaces can be ground and reincorporated, at a limited percentage, into new hot-mix asphalt surfaces. Recycled asphalt can also be used as base course or as road surfacing on unpaved secondary roads to prevent dust and improve performance (Hack and Bryan, 2006).

The decision of whether to recycle concrete or asphalt or dispose of it in landfills is usually based on contract terms, legal mandates, and economics. The decreasing availability of landfill space, tipping fees (the cost of depositing material in landfills), and environmental concerns have stimulated the recycling of asphalt and concrete-bound aggregate. The future for recycling aggregates will be influenced by landfill availability, greater product acceptance, government recycling mandates, increased availability of demolition material for recycling, and the demand for the sustainable and wise use of resources in the economy (Wilburn and Goonan, 1998).

9.5 Performance of aggregate in use

High-quality aggregate used in an unbound state commonly will last indefinitely. The performance of aggregate used in a bound state, such as in cement concrete or asphalt, is dependent on a number of factors including the physical and chemical properties of the aggregate, the specifications for use, preparation of the blended product (asphalt or cement concrete), method of emplacement, and care taken during emplacement.

The physical and chemical properties of aggregate are a result of its geologic origin, mineralogy, and subsequent alteration and weathering. The properties that are important depend primarily on the application of the aggregate (Langer and Knepper, 1998; Smith and Collis, 2001). Aggregate should be strong enough to support the intended load; should resist mechanical breakdown resulting from the action of mixers, mechanical equipment, and traffic; and should be able to withstand stresses caused by repeated freezing and thawing, or wetting and drying. Aggregate to be used in the manufacture of cement concrete should not contain minerals that cause adverse chemical reactions with the cement or with the steel reinforcing bars in the cement concrete structure. Aggregate used in asphalt should have the proper electrochemical properties to allow it to adhere to the bitumen.

Organizations in Australia, the European Union, the USA, and many other countries have established specifications for the use of aggregate and have designed engineering tests to determine compliance with specifications. These tests commonly expose aggregate to conditions that simulate the conditions under which the aggregate will be used. Specific requirements commonly are determined by the users of the material, which include federal, state, county, and city governments, and private industry. Aggregate used in road building and concrete construction is subject to very rigorous specifications, but these specifications, as well as the specifications for other applications, vary from area to area.

9.6 Waste products from aggregate mining and processing

Mining and processing of aggregates commonly result in the unintentional production of waste products, often with no readily available market. Processing wastes, also referred to as quarry fines, are fine-grained material resulting from crushing and screening. They generally are inert and nonhazardous, but commonly are difficult to handle, especially when wet, and are prone to movement by water, wind, and gravity. Production of fines ranges from 15% to 40% of the material being processed.

Most fines can be used in low-specification applications including fill, walkways, surfaces for horse tracks, and so forth, and to a limited degree in higher-specification applications including hot-mix asphalt and cement concrete. Limestone fines have a number of special applications including use in animal feed, in concrete blocks, as cement replacement in the production of concrete, for flue gas desulfurization (Hudson et al., 1997), and possibly for use as an agent to sequester CO_2 by accelerated weathering of limestone (Langer et al., 2007).

9.7 Sustainability of natural aggregate

Natural aggregates are the major ingredients in concrete and asphalt and as such are the mainstay of our infrastructure. They are cheap and readily available. There is no convenient substitute for the function they serve, and the demand for aggregate cannot be met from recycled products alone. If we wish to maintain our current lifestyle and pass the ability to maintain that lifestyle on to our progeny, we will need huge amounts of natural aggregate.

Aggregates, like all mineral resources, are a “wasting asset.” Eventually individual deposits become depleted, pits and quarries close, new deposits must be found, and new pits and quarries will be opened. Although the worldwide supply of potential aggregate resources is nearly infinite, potential sources of aggregate exist only in specific geologic environments and are not always where we need them. As examples: The Netherlands lacks hard rock resources suitable for the production of crushed stone; Austria does not have broad alluvial lowlands and has a shortage of sand and gravel (ECO-SERVE Network, 2004); geologic sources of coarse gravel and high-quality rock for crushed stone are very limited in the USA in the Gulf Coastal Plain, the Colorado Plateau, the Wyoming Basin, and the Great Plains (Langer, 2002a).

Even if sources of aggregate are present, the aggregate must be of sufficient quality to be put to use. Quality parameters can restrict the development of some aggregate resources. Many easy-to-locate resources have already been mined. Urban expansion, zoning, encroachment by incompatible land uses (referred to as sterilization), and citizen opposition can further limit production of aggregate. Large parcels of land have been divided into small parcels, and dealing with multiple owners has confounded the purchase of aggregate properties. It is not uncommon for producers to take 10 years to bring new supplies of aggregates on line (Wagner et al., 2002; Bleischwitz and Bahn-Walkowiak, 2006). Aggregate is heavy and bulky, and transporting it 30–50 km can double its price. The longer haul distances also result in higher rates of traffic accidents, more greenhouse gas emissions, and increased road and vehicle maintenance.

SARM is an appropriate framework for addressing these complex issues associated with aggregates development (Shields and Šolar, 2004). It is an approach that supports the development of policies that reflect good science, public preferences, and financial and social constraints (Šolar et al., 2004). SARM can be organized according to the environmental, economic, and social dimensions of sustainability.

9.7.1 Environmental value and responsibilities

9.7.1.1 Environmental value

SARM has tremendous potential to improve our quality of life, create additional wealth, and restore the environment. In today's expanding suburban areas, recently mined-out aggregate pits and quarries, as well as abandoned sites, are routinely converted into beneficial second uses; often these uses replicate natural conditions (Fig. 9.7) and create biodiversity (Langer, 2003; Minerals and Nature, 2007). In addition, aggregate is used for a number of environmental applications including flue gas desulfurization, lake and watershed liming, acid mine drainage abatement, landfill construction, treatment of water and wastewater for municipalities and industry, and for erosion control (Moulton, 1991; Remick, 1991).

Fig. 9.7 Sand and gravel pit reclaimed as natural wetlands.

9.7.1.2 Environmental responsibilities

SARM requires the development of aggregate resources in an environmentally responsible manner that does not result in long-term environmental harm, even if short-term environmental impacts are unavoidable. Three environmental principles generally apply: the precautionary principle, the polluter pays principle, and eco-efficiency.

The precautionary principle states that we should not take actions when sufficient information is not available, if those actions have a high probability of causing significant environmental damage. The polluter pays principle requires the cost of a quarry to include funding of reclamation and remediation of negative impacts within the quarry and over the mine life cycle including after-care. Eco-efficiency requires efficient exploitation of reserves and resources (Šolar et al., 2004).

There are many voluntary and regulatory tools that can be used to control environmental impacts. These include environmental impact assessments, best management practices, environmental management systems, environmental accounting, environmental reporting, monitoring, and life-cycle analyses (LCAs).

9.7.2 Economic value and responsibilities

9.7.2.1 Economic value

Employment in urban and suburban areas is commonly defined by the workplace and transportation structures, which are comprised largely of aggregate. The natural capital embodied in aggregates is transformed into economic capital derived from the profits from the sale of aggregates. The physical economy grows at an estimated rate of 10 tons per capita per year (Bringezu, 2002). In other words, each year this amount of material, which consists mostly of aggregates and downstream products like asphalt and concrete, is being added to new buildings and infrastructures.

As aggregate flows throughout the economy, the "value added" multiplies repeatedly. For example, each step of the process of extracting and processing aggregate, incorporating aggregates into concrete, and pouring and finishing concrete for a building, bridge, and so forth, adds to the economy through sales and salaries.

9.7.2.2 Economic responsibilities

There are four main economic responsibilities embodied in SARM: (a) providing aggregates to meet the material requirements of society; (b) maintaining a viable business environment; (c) encouraging value-added production and employment; and (d) employing full cost accounting while remaining competitive. The first three of these are the responsibility of government; the fourth is the responsibility of the firm (Šolar et al., 2004).

Meeting the material needs of society involves ensuring that sufficient aggregate resources are available to the marketplace. This requires the identification and protection of sufficient reserves and resources, provision of land access, creation or maintenance of production capacity, and development and maintenance of infrastructure (transportation and energy networks). All these issues are interconnected and need to be balanced by policy makers and resource managers (Šolar et al., 2004). Unfortunately, the identification and protection of aggregate resources is generally not well understood or integrated into the planning framework (Wernstedt, 2000; Richards and Peel, 2003; Baker and Hendy, 2005).

Aggregate businesses need to remain competitive to stay in business. Maintaining a viable business environment requires a stable and feasible permitting regime; consistent application of rules and regulations; functioning capital markets; reasonable levels of taxation; and well-defined property rights (Šolar et al., 2004).

Development of value-added manufacturing (such as ready-mix operations, asphalt plants, prestressed concrete panels, and concrete pipe and block manufacturing) is an important economic aspect of SARM. The presence of a value-added sector can reduce the need for imported materials while allowing the local economy to capture the economic benefits (profits, employment, tax revenues) that would otherwise accrue in another region (Šolar et al., 2004).

Aggregate businesses have a responsibility to accept the full cost of operation, including costs of prevention or remediation of environmental damage. When all the costs are taken into consideration, some quarries will not be viable economic enterprises. However, firms can increase competitiveness by following best management practices, maintaining a well-trained workforce, modifying production processes, and upgrading product quality. Product quality can be an important market element that can be labeled and traded (Šolar et al., 2004). Quality can be achieved through voluntary quality assurance/quality control procedures, such as adherence to ISO 9000 requirements (ISO, 2007).

9.7.3 Societal value and responsibilities

9.7.3.1 Societal value

The infrastructure necessary to build and maintain the social systems of developing or developed countries cannot be created or sustained without aggregate. Paramount among the components of the infrastructure system is transportation. Simply put, the workforce and material necessary to maintain a healthy economy and social system cannot reach the market without an efficient transportation system.

The regional importance of the aggregate industry as a source of employment can be substantial. Each quarry job may result in the creation of four or five other jobs including subcontractors for various parts of the quarry operations, transportation, equipment manufacture and repair, and downstream users of aggregates such as the concrete ready-mix and asphalt operations (Lafarge, 2007).

9.7.3.2 Societal responsibility

The aggregate industry exposes workers to potential hazards, and reduction of operational risk is an essential part of aggregate extraction and processing. Occupational health and safety issues are commonly addressed through training programs, monitoring, health screenings, and by following best management practices. Identifying the values, interests, and goals of stakeholders is a necessary step to resolve the complex social issues of SARM. For example, the benefits of aggregate development are dispersed over very large areas, but the community where extraction occurs suffers most of the adverse consequences of resource development. SARM depends on fairness to those living near or impacted by quarrying while considering the regional benefits from aggregate extraction (Šolar et al., 2004).

9.7.3.3 Corporate social responsibility

Corporate social responsibility (CSR) is an integral part of SARM. Companies that practice CSR commonly define themselves as being: accountable to stakeholders; responsible for social, environmental, and financial performance; accountable everywhere they do business; and open to external codes of conduct. Such companies demonstrate their commitment to CSR by instituting standards and goals at all levels of the organization (WBCSD, 2000; Dunnett, 2004). CSR can increase long-term business viability, including growth and profits, and sends a signal to stakeholders that the company is a responsible corporate citizen (Shields et al., 2006), which can help companies acquire the “social license” to mine.

9.7.4 LCA of aggregate operations

From mining and processing perspectives, the net environmental, economic, and societal contribution of aggregate extraction should be positive over the entire life of the aggregate operation. A LCA can be used to assess aggregate operations by including exploration, project development, resource extraction, material processing, storage, and reclamation. LCA should also take into consideration the fact that aggregate resources provide potential benefits or consequences that can far outlast the mining life cycle, and should include assessment of the long-term impacts as well as the life cycle of the aggregate operation itself. On the one hand, for example, an aggregate operation may last 10 years, while the building the aggregate was used to construct may last for 100 years. Similarly, the postclosure benefits from the reclaimed site may last indefinitely. On the other hand, potential environmental impacts such as those to the groundwater may last well beyond the life of the operation if they are not properly addressed.

9.8 Status of sustainable aggregate resource management

A stringent policy on SARM does not exist in most countries. However, governments of some states, provinces, and territories in Australia, Canada, and the USA, some of the member states within the European Union, and some other countries have elaborated sustainable mineral resource management policies that recognize the broader minerals and mining sector. A few of those policies (eg, DETR, 2000) recognize the aggregate industry as a key sector contributing to jobs, a high quality of life, and wealth for its citizens. Some of the policies identify actions that can be undertaken to help industry meet society's demand for aggregates, although most of the policies regulate only the aggregate extraction and production processes (Langer, 2002c; Bleischwitz and Bahn-Walkowiak, 2006).

There is a slow but inexorable move by the aggregate industries toward implementation of SARM principles (Langer et al., 2003a; Bleischwitz and Bahn-Walkowiak, 2006). For example, the European Aggregates Association (UEPG) encourages, endorses, and practices sustainability (UEPG, 2007). Some member-state associations of the UEPG have their own sustainability initiatives (eg, Quarry Products Association, 2006). Some aggregate producers, especially large multinational companies, have voluntarily issued sustainability policy statements and annual sustainability reports, and made those documents available via the World Wide Web. Some companies have embraced the principles of CSR even though they have not elaborated their policies using sustainability terminology. Sustainability tends to focus on the closure of mining operations (Barker and McLemore, 2005) and community relations.

9.9 General approaches to sustainable aggregate resource management

There is no specific process that must be followed when applying SARM, but, in general, the process followed is iterative and consists of a number of hierarchical steps (Fig. 9.8). These steps may include, for example: issuance of policy statements, elaboration of objectives, establishment of actions, identification of indicators, and monitoring.

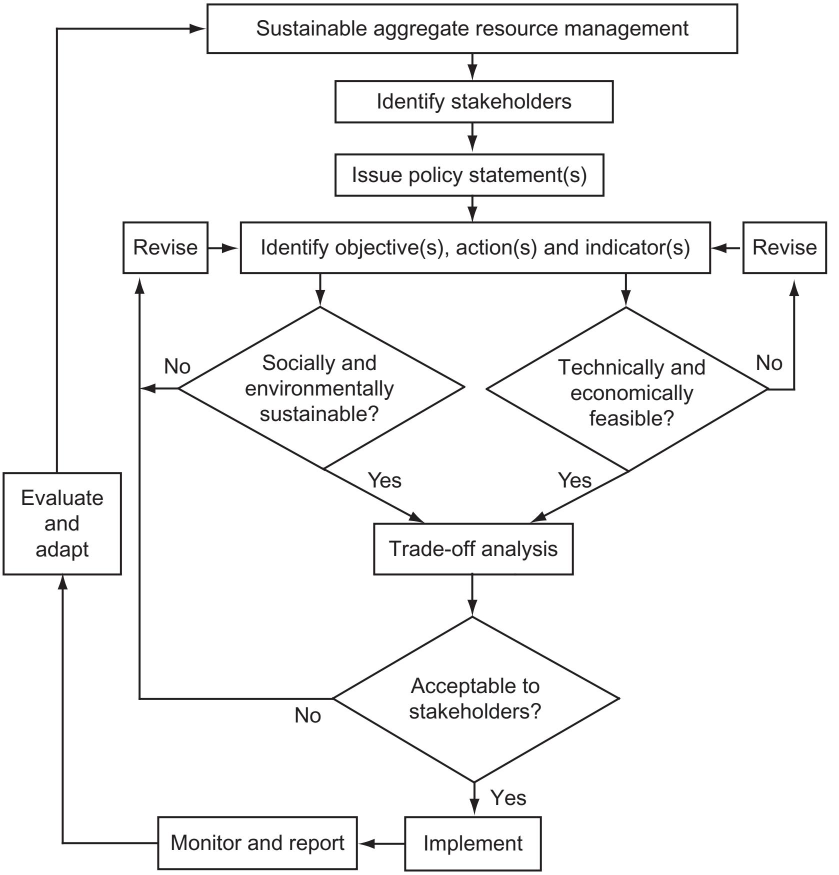

Fig. 9.8 Implementation process for sustainable aggregate resource management (SARM). Modified from Shields, D.J., Šolar, S.V., 2004. Sustainable Mineral Resource Management and Indicators: Case Study Slovenia. Geological Survey of Slovenia, Ljubljana, Slovenia.

Policy statements issued by governments commonly identify the aggregate industry as a key industry contributing to jobs, wealth, and a high quality of life for its citizens. The policy statements commonly commit the government to estimate the need for aggregates; assess the availability of aggregate; protect critical aggregate resources; and protect citizens from the unwanted impacts of aggregate extraction. Industry policy statements commonly identify environmental and societal concerns and commit the company to CSR.

Objectives describe what is intended to be accomplished. Actions are associated with objectives and describe the approach taken to reach the objective. Examples of paired objectives (in italics) and actions (modified from Plant and Haslam, 1999) are given in the following.

- *Maximize availability of, and access to, aggregate*—by forward planning that protects important resources from sterilization; by extracting as much aggregate as possible from an

area and using it for the most valuable application appropriate for the aggregate quality; by avoiding high grading (picking the best parts of the resource and limiting the ability to utilize the remainder); by finding uses and markets for all of the mined material; and by encouraging use of substitutes and recycled aggregate.
- *Minimize societal impacts and maximize societal benefits*—by forward planning that separates incompatible land uses; by creating community benefits for areas impacted by aggregate development; and by involving the local community in planning activities, expanding community awareness, and outreach. Minimizing societal impacts may lead to community acceptance and a "social license" to mine, which can be just as important as the legal permits.
- *Minimize environmental impacts*—by following best management practices and employing environmental management systems to identify and control potential impacts from aggregate extraction and processing; and by providing for conservation of natural surroundings by management of buffer areas that maintain or enhance vegetation and wildlife habitats and corridors.
- *Maximize rehabilitation of disturbed areas*—by reclaiming abandoned sites; by allowing for reclamation as an integral part of the quarry/pit design process; by following progressive, segmental, or interim reclamation processes where possible; and by being flexible enough to allow for advances in technology and changing local needs.
- *Identify and resolve legitimate concerns*—by constructively contributing to a decision-making process that addresses not only the interests of individual stakeholders, but also a wide range of objectives and interests of others.

Indicators measure progress toward reaching objectives and the effectiveness of actions taken. Indicators are specific to the target and actions, but tend to be similar in many situations. Examples of indicators from a government perspective (DETR, 2000; Langer et al., 2003a) include:

- the amount of aggregate produced compared with estimated production;
- the volume of material produced compared with the surface area converted for extraction;
- the proportion of aggregate coming from areas preferred for extraction;
- the proportion of aggregate coming from environmentally sensitive areas;
- the proportion of natural aggregates compared with recycled material;
- the proportion of sites covered by modern operating conditions;
- the area of reclaimed land compared with the area of land undergoing extraction.

Examples of indicators from an industry perspective (European Commission, 2006) include:

- the number of hours of training as a percentage of total work hours;
- the working time lost from accidents as a percentage of total hours worked;
- the total number of events arranged for neighboring communities;
- the energy consumption per ton of saleable product;
- the total land area in operation as a percentage of saleable products;
- the total number of reportable environmental incidents.

Monitoring, feedback, and the regular reconsideration of requirements as events develop help to refine the SARM process. Establishment of a joint monitoring process presents an opportunity to forge partnerships with communities and involve citizen groups.

9.10 Case studies

The following are condensed descriptions of case studies chosen to illustrate the variety of actions that have been taken as part of the sustainable management of aggregate resources. More detailed descriptions can be found in the associated references.

9.10.1 Government actions for resource protection and environmental restoration

9.10.1.1 Province of Modena, Italy, intraregional plan for extractive activities (Langer et al., 2003b)

The province of Modena, located in the Emilia Romagna Region in northern Italy, recognized that natural aggregate is necessary to sustain the economic well-being of the region. Modena Province prepared a variant of the intraregional plan for extractive activities (IPEA) that had been in place for about 10 years. Two objectives of the variant of the IPEA were to: (a) minimize the impacts from quarrying, and (b) guarantee the reclamation of quarries in a manner consistent with the existing landscape. In order to accomplish those objectives, the Emilia Romagna Region developed the innovative concept of the *polo estrattivo* (extractive district), which is not just one or more quarries, but is the whole area characterized by the prevalence of quarrying, including the intervening and surrounding territory that is subject to quarrying impacts (Langer et al., 2003b).

The variant of the IPEA encourages the management of quarrying through the extractive district. Aggregate can efficiently be extracted by mining aggregate between adjacent quarries within a district, or by deepening the pit, if that can be accomplished without harming the quality or quantity of the groundwater. The extraction of additional aggregate from abandoned quarries within the extractive district is encouraged. Both approaches could result in the extraction of more aggregate without disturbing more land surface area outside the extractive district. Previous aggregate extraction activities in rivers and floodplains, and channel modifications for flood control, have degraded the environment. Future quarrying in these already disturbed areas, followed by reclamation to natural habitat, is encouraged to make a positive contribution to biodiversity.

9.10.2 Government and conflict resolution

9.10.2.1 Management of aggregate resources in the Calahoo-Villeneuve region, Alberta, Canada (Richards and Peel, 2003)

A conflict of interests between rural citizens and the extractive industry resulted from the development of aggregate resources in the Calahoo-Villeneuve region of Alberta, Canada. Citizen opposition resulting from noise, dust, truck traffic, and concerns about environmental pollution, compounded by a lack of compensation by the aggregate industry to the local community, threatened to close access to regionally important aggregate resources. A government-led development of an area structure plan (ASP) implemented a number of programs to satisfy the concerns of residents while safeguarding access to aggregate resources. The plan included a voluntary levy on aggregate production, proceeds from which are paid into a Community Enhancement Fund. In general, the plan was well developed and executed, and it resolved the local concerns. However, the aggregate resource inventory for the area prepared by the Alberta Geological Survey was not utilized in the ASP, resulting in the sterilization of approximately half of the regional resources. When the aggregate resources identified in the ASP are depleted, the additional cost to haul materials from more distant deposits will be about CDN $1.6 billion (2003 CDN$). The county will forego approximately CND $45 million in contributions to the Community Enhancement Fund that would have resulted from development of the resources. Additional road hazards and maintenance costs will occur along with the production of about 682,000 tons of additional CO_2 in exhaust emissions.

9.10.3 Corporate social responsibility

9.10.3.1 Quarry expansion and community involvement (Langer, 2005)

A multinational aggregate producer proposed to expand its reserves in an operating quarry in a county near Denver, Colorado, USA, by exchanging company-owned land for existing, county-owned, dedicated open-space land adjacent to their quarry. The county is home to four of the five crushed stone operations in the Denver region. Crushed stone comprises 30% of the aggregate produced in the area and plays a major role in regional aggregate resource needs.

A similar proposal submitted by a different company about 10 years earlier had been denied. The new proposal was predicated on public trust whereas the earlier proposal was predicated on public relations. The company with the new proposal had earlier established a strong, long-term, favorable presence in the community. They openly, consistently, and effectively communicated their business plan to all stakeholder groups, and were visible and accessible.

The local government had no sustainability policies, and was not accustomed to facilitating industry/community interactions. Consequently, the company not only assumed their CSRs, but they also assumed the role of facilitator to encourage and enable other stakeholders to resolve legitimate concerns regarding the quarry proposal. The company successfully presented an enlightened proposal where the county ultimately gained 745 acres of new open-space land in exchange for 60 acres of existing open-space land adjacent to the quarry. The company doubled the life of its quarry, secured a location for a ready-mix plant and an asphalt plant, and eliminated the need to start a new quarry at an undeveloped location. The process involved collaborative efforts by all stakeholders and resulted in an outcome that balanced the needs of society, the environment, and business.

9.10.4 Industry and transportation issues

9.10.4.1 Transporting aggregate products in Derbyshire, UK (Aggregate Industries, 2007)

A large, multinational aggregate producer installed satellite tracking devices into a fleet of 30 tipping trucks within their Derbyshire, UK, operations. This initiative resulted in a number of specific benefits:

- improved customer service by providing real-time, accurate information on deliveries;
- improved vehicle utilization and performance by reducing the occurrence of empty return journeys and turnaround time of trucks, resulting in the potential use of fewer vehicles to move more materials, benefiting all road users;
- improved communication between sites and the vehicle, providing advance warning of the arrival of a vehicle to pick up product, thus reducing idle time, energy consumption, plant costs, and emissions;
- improved ability to monitor vehicle speed and driver performance, providing improved enforcement of local agreements and legal requirements in routing deliveries; haulers are more accountable for their behavior and consequences, and communities have greater confidence in the company's ability to manage vehicles.

9.11 Future trends

Industry will be able to meet society's needs for aggregate. However, the industry already faces local shortages of aggregate, a situation that will occur more frequently owing to inadequacies of present policies, urban encroachment, and citizen opposition. Leaving the management of aggregate resources to chance is likely to result in unintended consequences such as further sterilization of resources, juxtaposition of incompatible land uses, undesirable environmental impacts, and negative impacts on traffic. Price increases, land use conflicts, and shortages may encourage some local or regional governments to become more proactive in protecting access to aggregate resources as the availability of local supplies dwindles.

The factors that can be controlled to reduce the demand for aggregate are limited. The consumption of local aggregate can be slowed through use of alternative resources, such as recycled concrete and asphalt pavement, and by importing aggregate from other areas. The local aggregate supply can be expanded through technological advances and beneficiation of lower-quality aggregate, and in some instances through underground mining. Aggregate consumption, in general, can be slowed through the modification of application designs to require less aggregate or by modifying specifications to allow the use of lower-quality aggregate in certain low-end uses.

The application of SARM will advance, spurred on by green building, citizen opposition, and recycling mandates. Companies that practice SARM will appreciate advantages in obtaining their “social license” to mine. However, the aggregate industry consists of thousands of companies, and many will choose not to implement sustainability.

9.12 Sources of further information and advice

9.12.1 Information on the aggregate industry

Aggregates (Smith and Collis, 2001), *Sand and Gravel Production* (Littler, 2000), and *The Aggregate Handbook* (Barksdale, 1991) are comprehensive descriptions of the aggregate industry. The 7th edition of *Industrial Minerals and Rocks* (Kogel et al., 2006; Langer, 2006a,b) and previous editions, and *Geology of Construction Materials* (Prentice, 1990) contain chapters on the geology of aggregates. Six collections of papers describe global issues related to aggregate resources: *Aggregate 2001* (Kuula-Väisänen and Uusinoka, 2001), *Aggregates* (Primel and Tourenq, 2000), *Aggregate Resources: A Global Perspective* (Bobrowsky, 1998), *Aggregates: Raw Materials' Giant* (Lüttig, 1994), the *Proceedings from the International Symposium on Aggregates* (International Association of Engineering Geology, 1984), and *Natural Resources in the Geological Environment* (Kelk, 1992). Environmental impacts from developing aggregate resources and methods to limit those impacts are described in *Aggregate and the Environment* (Langer et al., 2004) and "Environmental impacts of mining natural aggregate" (Langer, 2001a).

9.12.2 Information on sustainable aggregate resources management

There are relatively few readily available published papers specifically addressing sustainability relative to aggregate resources; most are cited in this chapter. Key papers include: "Important features of sustainable aggregate resource management" (Šolar et al., 2004); "Planning for sustainable construction aggregate resources in Australia" (Baker and Hendy, 2005); *Sustainable development in the European aggregate industry: A case for sectoral strategies* (Bleischwitz and Bahn-Walkowiak, 2006), *Sustainable development in the European aggregates industry: For the benefits of future generations* (UEPG, 2007); and "Sustainability indicators for aggregates" (Langer et al., 2003a).

The European Aggregates Association (http://www.uepg.eu) and the United States National Stone, Sand & Gravel Association (http://www.nssga.org), as well as many of their other members, describe their SARM efforts on the World Wide Web (http://www.uepg.eu). Information about SARM efforts of some of the larger individual aggregate producers can be found by visiting corporate web pages. The Industrial Minerals Association—Europe (http://www.ima-europe.eu) and the Industrial Minerals Association—North America (http://www.ima-na.org) have posted descriptions of sustainability from a broader industrial minerals perspective on their web pages.

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The sustainability of lightweight aggregates manufactured from clay wastes for reducing the carbon footprint of structural and foundation concrete

10

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> *For instance, in London for Crossrail 1*[1] *tunnelling resulted in about 4 million tonnes of clay waste being excavated and transferred about 100 km for disposal. Had this clay been defined and designated as a mineral resource and used to manufacture LWA, about 2.8 million tonnes could have been produced with a particle density of 700 kg/m*^{3} *[0.7 Mg/m*^{3}*], equivalent to almost 10.8 million tonnes of natural coarse aggregate. Had this LWA been applied in concrete at the density of 1800 kg/m*^{3}*, more than 9.0 million cubic metres of low carbon footprint structural quality concrete could have resulted…'*
>
> *[Extract 10.4]*

> **Notation:** *In this chapter the constituent materials for concrete will be based on the typical values for density as shown in* Table 10.1*. Particle density is denoted as [PD] and density as [D]—kg/m*^{3}*.*

10.1 Introduction

The density of concrete has always been of interest to civil and structural engineers, for example, either increasing it as a gravitational addition for dams or reducing it as a means of increasing the height of buildings. The Romans exploited the technique for

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[1] **Note 1:** Crossrail 1 is a 100-km route running east to west via tunnels under central London to be operational by December 2019, designed to increase the underground rail capacity by 10% and carry 72,000 passengers per hour at peak times.

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Table 10.1 Reference values for density

Product	Density (kg/m³)
Portland cement (CEM I)	3100 (PD)
Fly ash (BS EN 450 Class S)	2350 (PD)
Natural (graded) aggregate 20-4 mm	2700 (PD)
Natural fine aggregate -4 mm (sand)	2650 (PD)
LWA (clay) 16–12 mm (single size)—dry	700 (PD)
Normal concrete (fresh)	2400 (D)
Lightweight concrete (fresh)	1800 (D)

turning slaked lime (Vitruvius, c.20BC; BLM, 1959)[2] into a hydraulic binder by using the pozzolanic "powder" derived from the alumina/silicates of the volcanic deposits of pumice, tuff and/or scoria. This together with the larger particulates became the fine aggregate for mortar, which with the coarse particles became the aggregate for concrete, with a density of about 1600 kg/m³ (Oleson et al., 2004).

In Roman times in Italy, resources of natural LWA such as pumice, tuff and scoria were all readily available, being near to the surface as the result of extensive prehistoric volcanic activity. As particulate ashes these deposits were easily exploitable, particularly beneath Rome, but where these types of resource are locally unobtainable LWA needs to be manufactured. However, in the UK between the years 1960 to 1990 many factories were established to produce LWA (Owens, 1993). Most of these factories produced LWA suitable for concrete masonry (blocks), but furnace bottom ash (FBA)[3] a waste from the coal power industry.

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[2] **Note 2:** Slaked lime is a colloid of lime (CaO) and "free" water that has replaced the term lime putty, which was the term given to the form that building limes were traditionally used and before the introduction of (powdered) hydrated lime in the 20th century. It takes about 2 weeks to slake quicklime to lime putty, which was the technique used by the ancient Egyptians 5000 years ago and up to the 20th century for preparing lime into the hydrated form $Ca(OH)_2$. Heating lump calcium carbonate at >850°C decarbonates it to quicklime. When cooled in atmospheric air, quicklime is unstable and returns relatively rapidly to the previous carbonated form. Quicklime hence requires slaking (or hydrating) as soon as possible with water; in this process considerable heat is generated. Initially and before it is totally slaked, the mixture is not only unstable, but also very caustic, which makes it difficult to use. In application slaked lime retains water, reacts gradually and hardens relatively slowly with a calcined alumina/silicate or pozzolana, thus the absence of cold joints in Roman concrete construction. Throughout this chapter the term slaked lime or hydrated lime as $Ca(OH)_2$ is referred to as lime.

[3] **Note 3:** FBA is a waste particulate material from UK thermal power stations that has been extensively used as an aggregate for concrete blocks. FBA replaced furnace clinker from chain grate boilers phased out by the early 1960s (not to be confused with combustible "coke breeze" that had been phased out by the 1940s). During the 1970s about 70% of electricity was produced in the UK from turbines using pressurized steam raised in furnace boilers fired with pulverized bituminous coal containing 16–20% ash. This ash, produced under oxygen-reduced conditions, is known in the UK as pulverized fuel ash (PFA), produced as fly ash and FBA in the ratio of about 80:20. As furnace sizes have increased and in particular since the advent of the 500 MW and larger units together with the greater size of power stations built since the 1960s, the production of FBA has increased to about 2.5 million tonnes/annum. FBA when crushed and graded has a very vesicular and angular particle shape that has properties ideally suited as an aggregate that has been an appropriate and cheaper alternative to the manufacture of LWA for concrete blocks.

The FBA was found to be a suitable aggregate for the production of lightweight building blocks. However, subsequent to its extensive application in blocks over at least four decades and not until relatively recently, FBA has become subject to the requirements of a construction material standard. This standard, outside the jurisdiction of European normalization, has become subject to a UK scheme for a waste material, judged to satisfy the requirements of a quality protocol devised by the UK Environment Agency and supervised by the Waste and Resources Action Plan (WRAP, 2010). However, the omission of requirements for FBA to satisfy a formal British standard provided strong competition to an unstable LWA industry; hence the sustainability of a dedicated factory for the production of LWA has become a precarious proposition. Manufacture of LWA depends on local mineral resources that with time are either unreliable, too expensive or becoming exhausted together with properties that are often restricted for application in structural concrete. Unfortunately, most if not all of the production of LWAs manufactured in the UK described as available in 1993 (Owens, 1993) had ceased by 2010. As a consequence, the current industry manufacturing LWA suitable for structural application has developed into a small niche market. Climate change and new UK legislation will have a direct impact on the lightweight concrete block industry with the closure in prospect of UK coal-fired power stations by 2025 (Hansard, 2015). The consequence of the rundown of power stations during the next 10 years will gradually reduce the amount of FBA and fly ash available, although enough fly ash has been stockpiled for processing (UKQAA, 2015). The effect on the block industry will inevitably increase demand for alternatives such as LWA for blocks for adequate strength and insulation. Progressively, climate change legislation will inevitably not only increase the production of LWA for blocks for low thermal conductivity, but also the demand for structural lightweight concrete. Hence the sustainability of mineral resources needs to be secured, not only to overcome the obstacles described, but also to surmount the perception of greater costs for both LWA manufactured from preserved “waste” clay and for the reduction in cost of concrete construction.

As will be demonstrated later, LWAs are required that are designed for application in structural concrete in order to reduce the carbon footprint of buildings. Therefore resolution to mineral resourcing is paramount, together with the mantra of how a particular source of LWA can in application and in sufficient quantity, be applied for a wider range of concrete density and strength in structures. In consequence the availability of large quantities of clay, a soft rock of low-bearing capacity produced from large civil engineering projects such as Crossrail 1 *[Note 1]* in London, ought not to be considered as waste. This clay should be designated as a mineral resource for both the manufacture of LWA (Boarder and Owens, 2015) and probably also as ground calcined alumina-silicate for combination with Portland cement.[4] Adding value to extracted and appropriate clay from infrastructure and

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[4] **Note 4:** Reference will be made later to calcining clay for application as a cement constituent, but this is subject to other research that is not discussed here and is beyond the scope of this chapter.

the deep foundations of tall structures during construction would not only resolve the availability of "near-to-market" mineral resources and provide sustainability for LWA manufacture, but would also reduce the cost of construction. This in conjunction with combinations of coarse natural and LW aggregates (UK Patent, 2014) provides the necessary versatility for concrete density and strength for more widespread application in structural and foundation concrete. Further, to significantly reduce the carbon footprint and to protect the reinforcing steel encased by concrete requires a revolution in concept and values in order to maintain concrete at the forefront of sustainable construction.

10.2 Concrete development

The 19th-century concept for concrete was to compete with Portland stone masonry, thus the idiom Portland cement. The 20th-century perception of concrete was to extend its purpose to compete with structural steel, thus the need for greater concrete strength and impermeability. This presented a greater challenge, for once steel bar was used to reinforce concrete to improve performance in tension (NBC, 1956), the other properties, particularly the water/cement ratio of concrete, required greater attention. Generally this has resulted in continuous improvement to the manufacture of Portland cement, making it more efficient as a binder, both in terms of fuel saving and performance.

The hydraulic cement (binder) is often made to a specification that does not fit every application, particularly when in situ conditions are taken into consideration. Further, immense variation of the binder is permitted (BS EN 197-1: 2011), where the constituents, their composition and combinations are diverse, encompassing a vast range of properties and performance. Even then the final concrete is tested at the standard temperature of 20°C (BS EN 12390-2: 2009), rather than at the temperature developed in situ. This is where the energy contained in the binder is usually ignored, as the heat of hydration has significant and different effects on the performance of the hydrated binder. The quality and strength of the hydrate depends on the size of the concrete element (Bamforth, 1980), the aggregate type (Owens, 1982) and the way temperature affects the strength and permeability of concrete (Owens, 1985). When LWA is applied, the lower heat capacity and lower thermal conductivity of LWA provide attributes that can enhance the properties of structural concrete, but the binder used has also to complement these attributes.

However, late in the 19th century and before the general introduction of steel bar as reinforcement, concrete was used in traditional gravity structures such as for the Glenfinnan railway viaduct, Fig. 10.1 in Scotland (Paxon and Shipway, 2007). This structure built between 1897 and 1901 is a prime example of civil engineering enterprise, where the recovered debris from this railway's tunnels was applied as natural dense aggregate for concrete. The concrete for the viaduct also used Portland cement that was manufactured before the advent of either a British standard (BS 12, 1904) or the rotary kiln in the early 20th century (Davis, 1934). Nevertheless this concrete is

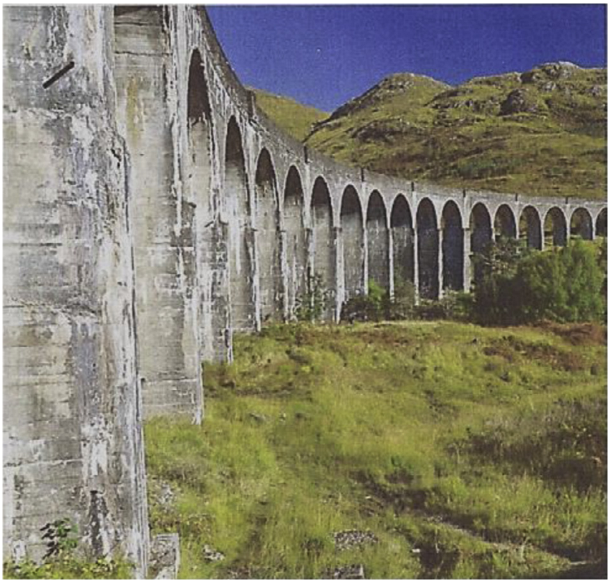

Fig. 10.1 Glenfinnan Viaduct. The West Highland Railway Fort William to Mallaig, constructed between 1897 and 1901.
Image from Walkhighlands.

Fig. 10.2 Inverted Tee-beam prestressed lightweight [Lytag] concrete bridge constructed in 1971, fresh density 1890 kg/m³, UK Forestry Commission, Redesdale Forest, Northumberland. Philip L. Owens, image 2012.

destined to last considerably longer than the 120-year working life required of current bridges. In the course of the early 20th century and once the steel-reinforced concrete beam was accepted in about 1906 (NBC, 1956), it was applied as the alternative to the arch and when improved by prestressing, where concrete entered into competition with the rolled steel joist (RSJ), such as in 1971 for the Redesdale Forest bridge (Fig. 10.2) in Northumberland (Higgins et al., 1983). As a consequence further attention to concrete properties has been required, where the physical and chemical properties of "fit for purpose" materials for concrete have become more closely interwoven and more complex. This complexity requires greater attention to the combined role of binder and aggregate, and the effects these have on the properties of structural concrete, such as microcracking (Forrester, 1965). Further, International obligation requires the environmental impact of concrete to be reduced and also its carbon footprint (ISO 14067:2014); this indicates that the density of concrete needs to be significantly reduced during the course of the 21st century. Hence, recognition of the performance of concrete during the course of history, and the achievements of concrete construction in the 20th century, becomes the signposts to a sustainable future for lightweight concrete.

In general both coarse and fine aggregate used for structural concrete amounts to about 70% of its volume; notionally concrete composition is subdivided into mortar and coarse (20-4 mm) aggregate; in principle, brickwork is also a form of concrete. With sources of natural aggregate (BS EN 12620:2013) dominating, little consideration, opportunity or even need until now has been given to modify their properties. Aggregates are size graded to encourage interstitial filling to make the best economic use of the binder. Essentially, for structural concrete about two-thirds is coarse aggregate with the particle sizes between 20 and 4 mm; this represents the constituent with the greatest influence on concrete density. The remaining one-third is fine aggregate (4 mm) required for the mortar. The amalgam of air, hydraulic binder, water and fine aggregate is mortar that occupies between 50% and 60% by volume of concrete. This amalgam equates to a combined density of about 2180 kg/m^3, representing about 1200 kg of a structural concrete's density. Traditionally in the UK, the coarse aggregate used for most concrete is typically from natural sources such as gravels (flint or quartzite) and crushed rock (limestone and granite) that occupy between 40% and 50% by volume of concrete. The particle density of natural aggregate is in the range 2600–2800 (mean 2700) kg/m^3, representing about 1200 kg of the concrete density. Hence, by combining the masses of mortar and natural coarse aggregate, the result is fresh concrete with a density of about 2400 kg/m^3; this of course varies slightly with location and the aggregate type available. Dense natural aggregates in concrete have the greatest influence on the rate of heat transfer (DSIR, 1959).

In the UK and in the period generally between the years of 1960 and 1990, there were many publications supported by both the institutions and engineers extolling the virtues of lightweight structural concrete (C&CA, 1970; Spratt, 1974; Concrete Society, 1978; FIP, 1978; IStructE, 1987). The advised reductions of at least 20% from the typical density of about 2400 kg/m^3 to between 1950 and 1650 kg/m^3, were in order to reduce construction costs, improve structural performance and concrete's image. An extract from (C&CA, 1970) reflects some of the UK's early history of lightweight structural concrete.

> *'The first building frame of reinforced lightweight aggregate concrete in Great Britain was a three-storey office block built at Brentford, near London, in 1958. Since then many structures have been built of precast, or in situ, prestressed[5] or reinforced lightweight concrete, thus indicating that lightweight concrete has the same wide adaptability as normal dense concrete.'*

In general the application for reduced density (lightweight) concrete in the UK has concentrated on superstructure slabs, beams, etc. Little attention has been given to its ideal application in the larger elements of basement slabs and walls. Whereas in 1969 for building in similar ground conditions as London, the 220-m-high 52-storey One Shell Plaza, Houston, USA, used lightweight concrete (1850 kg/m^3) throughout for nonpiled raft foundations (basement) and superstructure (Khan, 1969). This level of innovation in the UK is therefore possible, but has been disregarded probably due to the deceptively greater cost of the concrete/unit volume, and the lesser quantities of LWA available to make structural lightweight concrete a sustainable proposition.

Aspects of performance and value for money for the reduction of carbon footprint can now be better evaluated by requiring less concrete and by increasing the versatility of reinforced concrete with LWA. This is achieved by supplementing supplies of LWA that are increased by "aggregate combinations" (UK Patent, 2014; Boarder and Owens, 2015), which aids improvement to the performance of concrete used for larger elements, especially when fly ash (BS3892-1:1982, BS 3892-1:1993, or BS EN 450-1:2012) is part of the binder (BS 8500-2:, 2015). Then together with the benefit of lower density, that significantly controls and increases cracking capacity of concrete by first reducing both thermal conductivity and movement (Harrison, 1992) and second, with increased tensile strain capacity (Owens, 1982). These properties are further enhanced with lower permeability to water (Owens, 1985) when tested by initial surface absorption (BS 1881-208:1996). Further improvements to concrete are possible by mix design (FIP, 1983; Owens, 1986); this in turn can be related to the additional temperature rise that concrete undergoes in situ by the application of LWA (FIP, 1983). This method of temperature development and conservation increases the thermal dynamic of concrete that allows for the optimization of fly ash in combination with Portland cement (Owens and Buttler, 1980). When the full structural strength of lower density in situ concrete is not immediately required (MPA, 2015), and the practical aspect of exothermic heat in situ is factored in (having being previously ignored), the benefits of using fly ash can then be realised. Such materials used as constituents for the binder that are chemically reactive to the hydration products of Portland cement "clinker" *[see Note 4]* are more durable (BRE Digest 1, 2001). Hence the hydrates produce different criteria for the specification of in situ concrete, undeniably and unlike those hydrates produced by the quality control requirements for standard

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[5] **Note 5:** In 1969 a new grandstand at Doncaster Racecourse was built with precast prestressed lightweight concrete (density 1890 kg/m^3) in roofing units that cantilever 18 m above ground level to give an uninterrupted view of the racecourse and shelter to spectators.

20°C storage in water that is inappropriate for in situ concrete and inevitably leads to erroneous conclusions (Buttler et al., 1982) *[Note 6]*.[6] This is especially important for assessing carbon footprint (ISO 14067:2014) when concrete grade (strength) (BS EN 12390-2: 2009) is taken as that measured at 28 days after water storage at 20°C (MPA, 2015). Hence, when full advantage is taken of LWA and fly ash for the different structural applications of concrete, the combination of lower heat capacity and thermal conductivity are attributes that enhance concrete properties (Brooks et al., 1987). Thus, without requiring in situ concrete strengths to be much greater than 20/25 N/mm^2, but that can now be designed with enhanced durability with lowered permeability to water and gas, together with increased resistance to attack from chemically aggressive aqueous solutions (BRE Special Digest 1, 2001; BS 8500-1:2015), lower density concrete can be used for not only marine applications (FIP, 1978), but also for high-performance foundations.

This requires in the UK, for instance, coarse LWA of different properties than is currently manufactured (or found naturally), thus requiring LWA to be manufactured to order or imported. Sustainability then becomes an issue when the LWA is not designed for purpose for the following main reasons: unreliable resources, insufficient availability, restricted properties and higher costs. Consequently these impediments have prevented a fuller appreciation of how the thermal dynamics of binder hydration, in conjunction with coarse LWA, can change and improve the properties of in situ

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[6] **Note 6:** Since awareness in the early 1980s of the thermal dynamics of concrete (the rate of chemical reaction or hydration on exothermic heat, related to the conditions for this heat development and retention) and the effect that temperatures above 20°C have on the rate of binder hydration and the consequence this has on the physical performance of in situ concrete, design engineers have often found difficulty in relating these to the specification of concrete and in accounting in situ for these beneficial effects, especially when the temperatures generated above 20°C deleteriously degrade the strength and permeability of concrete made with 100% CEM I, while improving the strength of cement combinations (CEM I + fly ash) (Bamforth, 1980). Part of the challenge is to assess these effects before formulating a concrete specification and commencing construction, as both affect the selection of an appropriate concrete mix for foundations, thick slabs and walls. There is as yet no commonly accepted method to evaluate and assess the consequences of a particular mix on concrete performance. In an attempt to develop a method or protocol to assess concrete performance, without too much interference or damage to the hardened concrete (eg, cutting and removing cores), a protocol has been developed and described by (Boarder and Owens, 2015). The protocol involves the application of a 194-mm-diameter × 650-mm-long (and could be longer) cylindrical and insulated mould or pod (Fig. 10.3), that uses sufficient concrete to produce 2 cylinders that have undergone similar in situ rise of temperature and profile over 7 days as would be developed in a concrete element typical of at least 1.25-m minimum dimension. Such concrete when subjected to the conditions of the protocol can be tested for an upgraded measurement of in situ density (not based on 100-mm cube), also thermal movement and strain capacity, together with microcracking, permeability, elasticity and strength. Evidence also indicates that the greater the lime saturation factor ($C_3S : C_2S$ ratio) of a Portland cement clinker and the greater the temperature developed during hydration, the greater will be the proportion of $Ca(OH)_2$ produced (Buttler et al., 1982) for reaction with fly ash. The Roman success with concrete is based on the large size or volume of element used for their typical gravity structure, which equates to increasing the thermal dynamic of the binder. Also, by using LWA combined with the greater ambient temperature of Rome compared for instance to London (on average 6.5°C in summer) the thermal dynamic for concrete with a slow rate of hydration would be significantly increased.

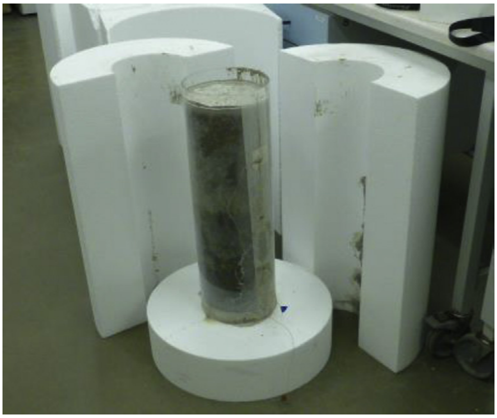

Fig. 10.3 The stripped down insulated mould or pod, showing the concrete encased in the inner acrylic tubular lining, which supported the thermocouples and other instrumentation.

structural concrete compared to that made with natural aggregate. In this instance LWA not only stimulates the rate of hydration and with it increased temperature in situ for more rapid hardening to increase production (FIP, 1983), but also the application of cement combinations (BS 8500-2:2015) particularly containing fly ash (or similar alumina-silicate) that requires temperatures in excess of 30°C to encourage the "pozzolanic" reaction (Owens and Buttler, 1980). The challenge therefore is to capitalise on the 2000-year-old knowledge (Vitruvius, c.20BC) that certain clays expand and make an LWA when heated >100°C above typical UK brick-calcining temperatures of 1080°C (Grimshaw, 1971). Further, while most contemporary LWAs produced from London(-type) clays can be used for the production of structural concrete, there are obstacles, such as high water absorption and not being best suited to the specification of a selected density, therefore inhibiting the wider application of lightweight concrete for efficient and cost-effective construction.

The water content of concrete is important, not only for consistence, but also for other properties, and its reduction increases the durability of concrete, and while water-reducing chemical admixtures contribute, fly ash complying with (BS 3892, 1993) helps by physically reducing the water content of concrete by about 8% (Owens, 1979). The sustainability of fly ash supply in the UK is dependent on the changes required by legislation in response to energy and climate change policies; the intention is for all unabated coal-fired power stations (those without carbon capture) to be closed and replaced by gas-fired stations by 2025 (Hansard, 2015). Should this occur, one alternative is to calcine clay in order to maintain the competitiveness of all forms of concrete; therefore research should be ongoing to meet this expected challenge and deadline. Since the mid-1970s fly ash, however, although considered a waste of no use to the power industry, has become a significant constituent of cement that has increased concrete durability at no extra cost to the UK. Fly ash is produced at poor efficiency (maximum 38%) thermal power stations that inject pulverised bituminous (low- calcium) coal into furnaces operating at about 1400°C. Fly ash is a powder of "glassy particles" of unique spherical shape that are removed by electrostatic precipitation from the gas exhaust that also contains volatized sulphur dioxide, the cause of acid emission which contributes together with CO_2 for climate change. UK fly ash is the residue of the noncombustible content of coal, when in geological time it was pervaded by alluvial clay at the time of formation and was transformed into shale that on being pulverized and fired melted into the small particles characterised as fly ash. Fly ash also contains cenopheres that are the result of "shale" expansion. The chemical composition of UK fly ash is about 80% alumina-silicate; the importance of this to lower density concretes is significant for reducing CEM I and the carbon imprint of concrete construction.

What is proposed addresses sustaining the wider application of reinforced concrete using LWA manufactured from waste clay, ceramically expanded and specifically intended to preserve and extend the other resource materials for concrete that deplete with the rate of construction. The considerations are

- For every structure, LWA in concrete saves nonrenewable resource materials in proportion to the concrete fresh density.
- Using appropriate excavated clays that are otherwise wasted not only reduces the cost of disposal, but also gives added value to the clay and reduces the rate of exploitation of nonrenewable resources.
- The technique of "aggregate combinations," using coarse natural and clay-based LW aggregate, allows for the specification of concrete density to between 2400 and 1600 kg/m^3. This can typically reduce the mass of concrete in a structure of between 20% and 30% and the volume of concrete by 10% to 20%.
- Waste clay is sustainable. It is produced particularly in cities such as London at a rate commensurate with construction. Should the rate of construction increase for the approximate conversion of 1 for 3 due to clay expansion, twice as much coarse LWA is made available. This increases construction productivity, without requiring any investment in "greenfield" nonrenewable resources.

The following section addresses the issues of lightweight aggregate manufacture to secure the future and sustainability of *specified density* structural lightweight concrete as a means for reducing the carbon footprint (ISO14067: 2014) for future reinforced concrete construction.

10.3 Lightweight aggregates

What came first, the chicken or the egg? The contemporary understanding of lightweight concrete usually comes from the iconic Roman structures of the Coliseum completed in c.80 AD and the Pantheon completed in c.126 AD. These were built from the deposits of volcanic ash particulates under Rome and that previously, but similarly had in 79 AD buried Pompeii with "ash rain" and Herculaneum with pyroplastic surge. These deposits of volcanic ash as natural alumina-silicate glasses relate to the enormity of the explosive eruption in prehistory that were exploited by the Romans for which the peninsula of Italy is renowned. However, it was not until the beginning of the 1st century BC that deposits of fine pozzolanic material and other coarser particulates of pumice and tuff found at Pozzuoli, on the coast of Italy near Naples, were first utilized for concrete. As plagiarists and as with most discoveries the Romans soon realised that these natural volcanic deposits when mixed with lime and water made an amalgam, now identified as a calcium-aluminium-silicate-hydrate (CASH) (Jackson et al, 2014), that was exploited to make a hydraulic concrete that started the Roman era of durable concrete construction. They first capitalised on this with numerous harbours around the Italian coast and then at locations around the Mediterranean (Oleson et al., 2004) on a relatively large scale, especially with concrete with a density of about 1600 kg/m^3 (Oleson et al., 2004), which because of the convenience of shipping became well established. This practise eventually contributed later to so many iconic buildings.

While all this was typical of Roman engineering enterprise, it was not until the discovery that subterranean Rome was also geologically similar to the Pozzuoli deposits and that mining could produce sustainable quantities of both pozzolana and aggregate. These volcanic deposits beneath Rome are mainly of tuff (alumina-silicate) from 20 mm down to dust and were described as rubble (Vitruvius, c.20BC), but were in such quantities that their building programmes could be sustained. Sources of calcium carbonate are also within about 10 km of Rome so there was sufficient lime for it to be applied in the concrete of such large elements that typify Roman gravity structures, such as the Pantheon.

Such structures were also possible where the formwork to arches, vaulted ceilings (Fig. 10.4) or domes could be left in place until the concrete had hardened sufficiently for the formwork to be removed. Today speed and the programming of construction time is of the essence and whilst for contemporary beams and floor slabs the early strength of concrete is important and a major consideration in project planning. In large elements time is not so critical for such things as retaining walls and basement slabs; while the rate of early hardening is increased with concrete using LWA (Fig. 10.5), the risk of cracking is significantly reduced also by the combination of

Fig. 10.4 The concrete of the vaulted construction in the Coliseum c.75 AD. Philip L. Owens collection 2003.

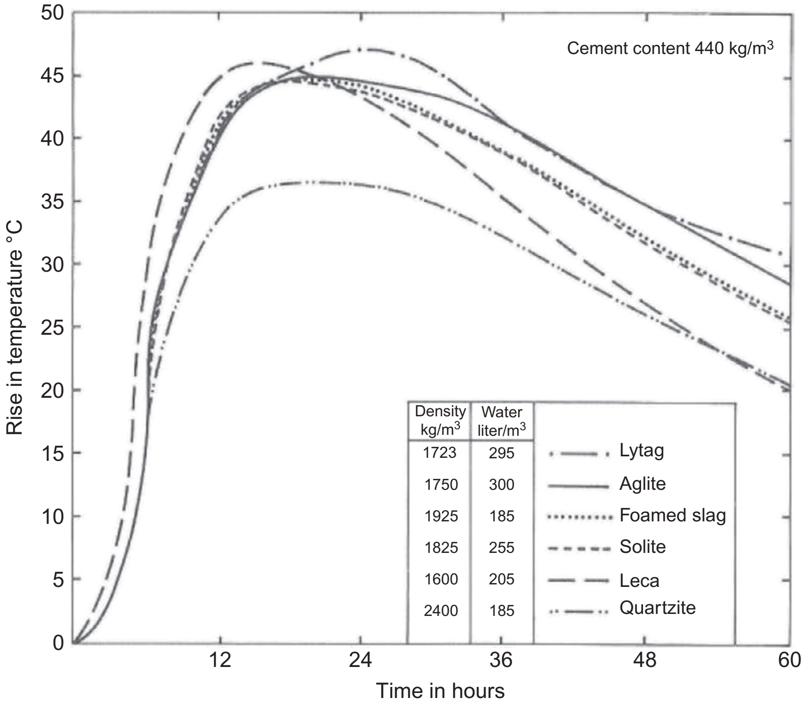

Fig. 10.5 The effect of coarse aggregate type on the thermal dynamics of concrete and temperature rise in an insulated 300 mm cube.
Reproduced from FIP, 1983. FIP Manual of Lightweight Aggregate Concrete, second ed. Surrey University Press, London, p. 86.

lower thermal movement and greater tensile strain capacity. Hence concrete with lower density can be applied not only with confidence of its properties, but also to significantly reduce the overall weight of in situ foundations that do not require the strength of concrete applied for smaller structural elements, for example, columns, beams and slabs.

In Britain with the absence of pozzolana or volcanic sources of coarse and fine aggregate, the Roman style of concrete was challenging. However, as shown in Fig. 10.6 concrete could be sustained with crushed brick, where in combination as coarse and fine aggregate, together with its ceramic “dust” required for the chemical reaction with lime (Vitruvius, c.20BC), has made very durable concrete. Evidence also of Roman hypocaust (short) pillars recycled in the walls of the Norman castle at Colchester, Essex, shows a chemical bond between brick and lime mortar. However, the Romans were resourceful; they also used pulverised glass and pottery, as well as pulverising calcined septaria that forms as large agglomerations in clay. Septaria was exposed by erosion along the east coast of the Thames estuary and was used in Britain extensively from about 55 AD in the construction of the temple podium in Roman Colchester, and also in Orford Castle, Suffolk, the coastal keep built between 1165 and 1175 for the English

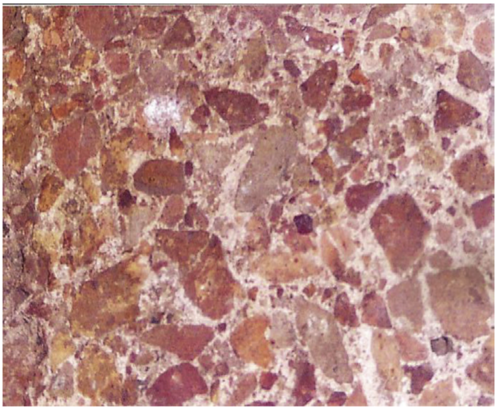

Fig. 10.6 The application of crushed Roman brick in concrete. Chesters Roman Fort and Museum, Northumberland.
Philip L. Owens collection 2012.

Plantagenet King Henry II and by the Rev. James Parker in the late 18th century for his patented Roman cement manufactured at Northfleet Creek, Kent, and at Harwich, Essex (UK Patent No., 2120, 1796). Nevertheless it was (Vitruvius, c.20BC) who had previously commented that *certain brick clays* when overheated expand and produce an aggregate similar to pumice, the principle which applies today for LWA manufactured from argillaceous minerals.

In 2015 LWAs were made from a variety of different resources, but we will focus here on clay-based LWA. Each LWA to be described here is available in the UK with its trade name and a description of its most appropriate application. The LWA most suitable for structural concrete derive from argillaceous resources[7] that expand when calcined to pyroplasticity between 1125°C and 1200°C.

UK fly ash has also been extensively used for the manufacture of LWA produced close to thermal power stations that inject pulverised bituminous coal into furnaces operating at about 1400°C. Fly ash is a glassy powder electrostatically precipitated from the gas stream exhaust of a power station furnace. The chemical composition of UK fly ash is as alumina-silicate glass, whose particles can be fused together at about 1100°C to form an LWA. Most LWA sources while produced to conform to the requirements of BS EN, 13055-1: 2002 have different applications, but probably structural concrete is most appropriately made with low water absorption coarse aggregate. LWAs with higher water absorption (>25% at 30 min) have particular application in concrete masonry and in the unrelated application of hydroponics.

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[7] **Note 7:** Besides clay in the UK there are deposits of other argillaceous minerals that have been exploited for the manufacture of LWA, for example, **aglite,** etc. (Owens, 1993) based on shale from coal mine tips and **solite** based on soft slates from the Horseshoe Pass, North Wales, and Ballachulish, in the Western Highlands of Scotland.

10.4 A brief description of the various lightweight aggregates available in the UK, their manufacture, properties and applications

Commercial quantities of LWA available in the UK in 2015 under trade names are listed in Table 10.2 and described below.

Techni·Clay (Fig. 10.7) is UK produced by Plasmor Ltd. Manufacture is based on expandable clay heated in an adapted but typical rotary kiln. The clay is first homogenised and then extruded and cut into green pellets, which “roughly” round off in a long rotary kiln. This LWA is manufactured specifically for lightweight masonry concrete blocks. Other applications include geotechnical fill, drainage and horticultural hydroponics.

Lytag (Fig. 10.8) recommenced manufacture in the UK in 2014 and is currently produced from fly ash produced by the coal-fired power station at Drax, South Yorkshire. This fly ash has been used in the UK for the manufacture of Lytag since the 1960s and has been the most frequently applied LWA in structural concrete. Depending on the sustainability of fly ash, the manufacture of Lytag is assured (see Section 10.2). Fly ash of less than 7% loss-on-ignition or “char” content (eg, unburnt fuel), with the volatiles removed as a consequence of firing under oxygen-reduced conditions, is mixed with some water and coal dust to about 8% fuel. This is formed into nodules of 8- to 14-mm diameter on a rotating dish noduliser. The nodules are transferred and fired on a sinter strand (a type of moving grate), through which combustion air is drawn. The nodules are heated to about 1100°C and while the fly ash is fused within the nodule, the process also slightly attaches the nodules to each other, so they have to be separated after sintering. The “sintered bed” of fused nodules is cooled by drawing air through and preheating it for fuel combustion. The bed of nodules is discharged into a “breaker” where the slightly fused nodules are separated. The finished particles of LWA are nearly rounded, but rough and open textured. Lytag has been extensively used in structural and prestressed concrete (see Fig. 10.2), but in the past has been characterised by entrapped water within the particle. This can make for two principal aspects in use, challenging consistence control for concrete production and, when hardened, the fire rating is lowered for enclosed spaces.

LECA (Fig. 10.9), the acronym for ***lightweight expanded clay aggregate*** developed in the 1930s, is of Danish origin and trademark. Although LECA can be made in a number of different ways, in principle it is an LWA manufactured from reserved resources of “expandable” clay. The excavated clay is first layer deposited in a “long shed” to roughly homogenise before being “scalped” as “chunks” that are conveyed to the kiln. Festoon chains at the rear of the rotary kiln reduce the chunks of clay into small nodules, which dry as they pass down the length of the rotating kiln angled at about 5° to the horizontal. With the rotation of the kiln the formed dry nodules progress until they reach the firing zone at a temperature of about 1125°C; while some vitrification of the particle surface occurs, the intention is to make an irregular/rounded shape, but mostly a smooth-surfaced aggregate of relatively high water absorption, for concrete masonry block manufacture.

Table 10.2 List of trade-named LWA available in 2015 to the UK

Aggregate trade name	Raw materials	Manufacturing process	30-min water absorption (typical) %	Dry loose bulk density (typical) kg/m^3	Sourcing	Country of origin
Techni · Clay	Expanded clay	Rotary kiln	15–25	500	UK	UK
Lytag	Sintered PFA	Sinter strand	10–20	825	UK	UK
Leca	Expanded clay	Rotary kiln	20–30	425	Imported	Denmark
Liapor	Expanded shale	Rotary kiln	20–30	550	Imported	Germany

Fig. 10.7 Particle density: 575 kg/m³; 30 min water absorption: 13.5%.

Fig. 10.8 Particle density: 1460 kg/m³; 30 min water absorption: 10.5%.

Liapor (Fig. 10.10) is of German trademark and origin being akin to LECA, in that while manufactured from an argillaceous resource of excavated shale in Germany, under licence in other countries clay is used. In Germany the shale is dried and pulverized to a fine powder and homogenised, before being made into controlled sized nodules on a rotating dish noduliser for firing in a shorter than normal rotary kiln. The nodules expand at a temperature of about 1150°C into mostly uniquely single-size spherical particles with some vitrification of the surface, thus offering some reduction to water absorption, but the finished product still has much greater water absorption than standard natural aggregates.

Fig. 10.9 Particle density: 595 kg/m³; 30 min water absorption: 18.5%.

Fig. 10.10 Particle density: 535 kg/m³; 30 min water absorption: 21.5%.

LWAs from both clay and shale can also be produced on a sinter strand; the resultant product is strongly fused and has to be broken and size reduced for application. LWA produced in this way is highly vesicular and not best suited for structural concrete.

This short appraisal of current manufacture of LWA available to the UK shows that while products are currently available they may not be focused on the properties of reinforced concrete to satisfy the requirements of the engineer, who at present adapts the superstructure design to suit the LWA source. The optimum physical requirements of LWA for structural concrete is to have a 30-min water absorption of between 5% and 10% in order to control the consistence of fresh concrete; a restricted particle size of between 10 and 16 mm for both best strength and avoidance of LWA floatation, with a spherical shape for the least amount of mortar. At present there has been no emphasis on the manufacture of LWA with properties specifically designed for structural concrete, presumably based on the assumption that "reducing concrete density implies lower concrete strength."

In the past, one of the principal challenges to the manufacturers of LWA is the lead time it takes for a structure designed with lightweight concrete to reach fruition and for there to be continuity and sustainability of LWA supply when construction starts. The manufacturers of all bulk products have to face the reality of sustainability and to ensure that resources available are sufficient to continue. Thus, while LWA manufacturers are no different, this can change with the application of sustainable resources of waste clay and "aggregate combinations" (UK Patent, 2014), a technique that encourages the wider application of lightweight concrete. Hence, concrete producers can stock a source of LWA and offer a range of concrete density to suit the required application.

In the past, the production of structural concrete of specified density has only in part been achieved. The density of concrete made with natural aggregates depends on the particle density of the coarse aggregate, which is usually between 2600 and 2800 kg/m^3, typically 2700 kg/m^3. As described earlier, the density of concrete depends on the particle density of the coarse aggregate with the resultant effect on the fresh concrete density varying between 2340 and 2460 kg/m^3, typically 2400 kg/m^3. Lightweight concrete in the UK and in the past has principally depended on the substitution of natural coarse aggregates, thus the particle density of the LWA selected dictates the concrete fresh density that on the same basis as that shown in Fig. 10.11 can be as low as 1600 kg/m^3, which in effect limits the standard 20°C water stored compressive strength to probably less than 30 N/mm^2 at 28 days (Spratt, 1974), but this is an oversimplification of the effects on other properties.

Due to the "standard" method of testing concrete at 20°C, *three significant properties influencing concrete performance are usually overlooked*. First, by progressively reducing density from 2400 kg/m^3, the thermal dynamics of in situ concrete changes (Fig. 10.3), increasing the rate of cement hydration. Second, this rise in temperature increases bonding at the interface of paste and LWA (Fig. 10.6) for greater strain capacity and third, the increase of in situ temperature especially promotes the pozzolanic reaction of fly ash, avoiding the detrimental and significant effect that increased temperature has on the strength of concrete made with CEM I alone (Bamforth, 1980). When fly ash is used as part of the binder, the resultant hydrate reduces the water (and gas) permeability of the hardened concrete (Owens, 1985). These properties are equally if not more important for the performance and appropriate application of lightweight concrete in structural design.

The application of the innovation of "aggregate combinations" (Boarder and Owens, 2015) graphically shown in Fig. 10.11, is where the fresh density of concrete can be specified and controlled between 2400 and 1600 kg/m^3. This technique enables

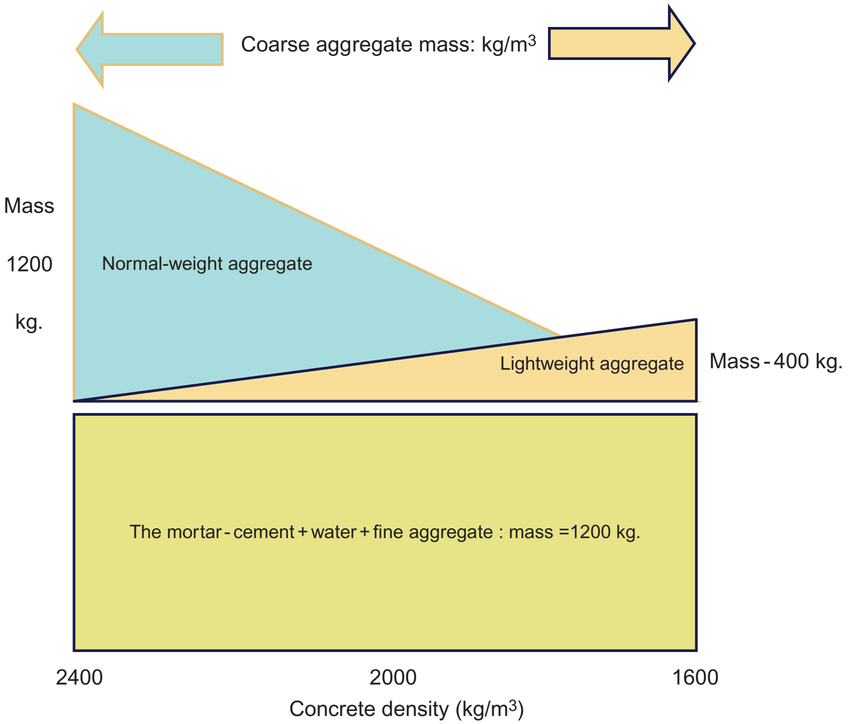


Fig. 10.11 The diagrammatic representation of the principles of coarse aggregate combinations.

management of fresh concrete density to that specified. For instance, a concrete fresh density of 1800 kg/m^3 requires 300 kg each of natural and LW aggregate, together with the mortar of 1200 kg/m^3. Fig. 10.12 is an example of coarse aggregate combination based on a 50/50- volume/volume-quartzite gravel and Liapor. Similarly density can be related to strength developed at 20°C by varying the cement content, by applying the techniques given in the Prime Mix Method (Owens, 1986). With these permutations of mix design for structural concrete, the carbon footprint of a structure can be significantly reduced, as not only does the concrete permit the mass of a building to be lighter, with ensuing cost savings, but also lifetime operating costs. For instance, in London for Crossrail 1, tunnelling resulted in about 4 million tonnes of clay waste being excavated and transferred about 100 km for disposal. Had this clay been defined and designated as a mineral resource and used to manufacture LWA, about 2.8 million tonnes could have been produced with a particle density of 700 kg/m^3 (0.7 Mg/m^3), equivalent to almost 10.8 million tonnes of natural coarse aggregate. Had this LWA been applied in concrete at the density of 1800 kg/m^3, more than 9.0 million cubic metres of low carbon footprint structural quality concrete could have resulted. Had only half of this been achieved, it would have made a considerable contribution to reducing the carbon footprint of new construction in London. To achieve this advance, expanded clay LWA manufacture is the necessary consideration.

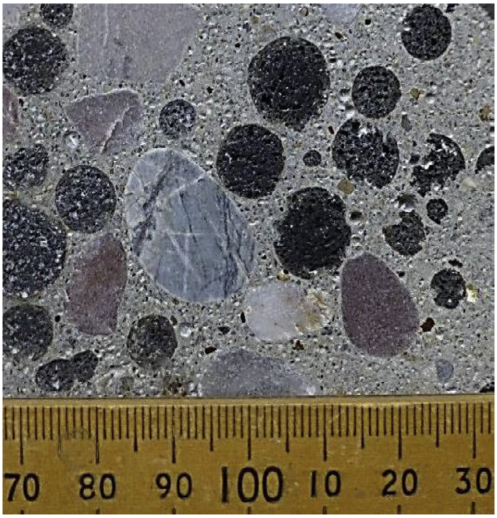


Fig. 10.12 The interior of concrete sawn to expose the coarse aggregate combination of quartzite gravel and Liapor. Concrete density 1925 kg/m³.

10.5 Expanded clay LWA manufacture for structural and foundation concrete

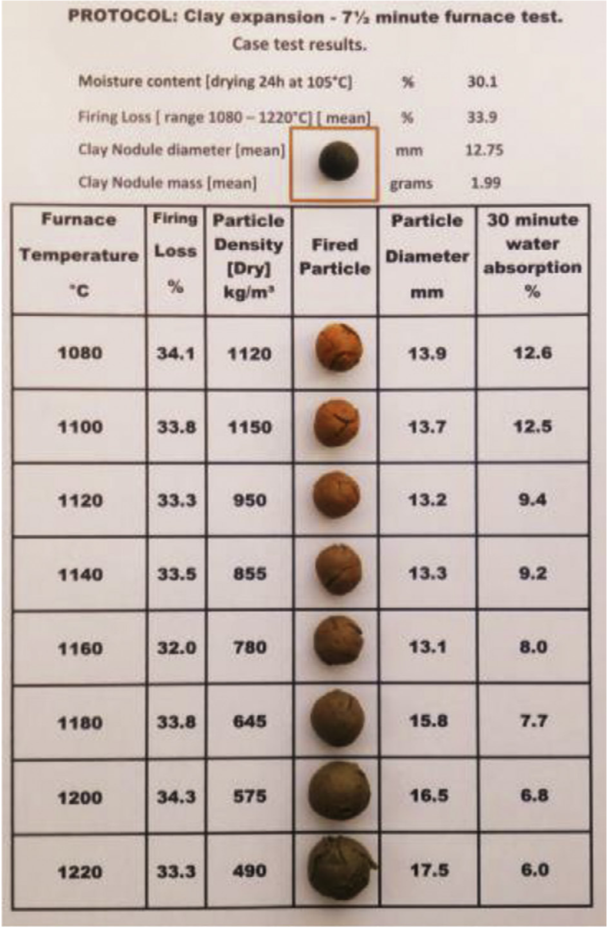
The protocol to test whether a clay source expands or not is assessed by preparing at least 4 nodules of about 12-mm diameter from freshly excavated clay. The nodules are prepared by hand rolling, weighed and placed on a 60-mm-diameter nickel tray to be heated at between 1150°C and 1200°C in a typical static muffle furnace (Fig. 10.13)

Fig. 10.13 Laboratory muffle furnace. Maximum operating temperature 1300°C.

for at least 7.5 min. Remove the "fired" nodules to cool to ambient temperature. Once cooled, weigh to determine the weight loss before placing the particles in a beaker of water for 5 min to observe whether they float. Continue to immerse for 25 min, but floating particles shall be fully submerged. After 30 min, remove and surface dry the particles to determine the water absorption and particle density. To inspect the degree of expansion, saw a particle in half to observe the internal cell structure; should the inspection display good pyroplastic expansion, continue with further testing. Fig. 10.14 shows the magnification of 20-mm-diameter expanded particles with vitrified exterior and the closed cell structure of the interior. Should particles that sink and not display any of these characteristics, it is unlikely and without the addition of organic matter that an internal structure of pore cells could be induced. These pore cells are probably interconnected, resulting in a water absorption greater than 10–15% and are thus **outside the scope of the tests to follow**. Hence with particles of the appearance and quality of those similar to Fig. 10.14, continue testing to determine the limits for expansion. For instance, with fresh clay normally containing between 22% and 28% moisture determined at 105°C, protect it to prevent drying and prepare nodules of 10/12-mm diameter for 7.5 min' exposure to different furnace temperatures. As shown in Figs. 10.15 and 10.16, by progressively increasing the furnace temperature at intervals of 20°C from 1080 to 1220°C the optimum temperature can be determined for the diameter of the "green" nodule to expand to a mean particle size of about 14 mm (±2 mm). Hence, correlation can be established between firing temperature, particle density and 30-min water absorption.

For example, after initial testing in the static laboratory muffle furnace, the waste clay from the tunnel excavations of London's Crossrail 1 tunnels was found to be ideally suited for the production of LWA as shown in Fig. 10.14, but the manufacture of LWA in a rotary kiln is entirely different to that of a static furnace. An expandable clay control of expansion relies on the kiln type and control of the kiln conditions, for as clay becomes pyroplastic and just reaches the point of significant expansion, an exothermic reaction occurs. This creates the vitrified coating to the surface of the

Fig. 10.14 An example of expanded London clay.



PROTOCOL: Clay expansion - 7½ minute furnace test.

Case test results.

Moisture content [drying 24h at 105°C]	%	30.1
Firing Loss [range 1080 – 1220°C] [mean]	%	33.9
Clay Nodule diameter [mean]	mm	12.75
Clay Nodule mass [mean]	grams	1.99

Furnace Temperature °C	Firing Loss %	Particle Density [Dry] kg/m³	Fired Particle	Particle Diameter mm	30 minute water absorption %
1080	34.1	1120		13.9	12.6
1100	33.8	1150		13.7	12.5
1120	33.3	950		13.2	9.4
1140	33.5	855		13.3	9.2
1160	32.0	780		13.1	8.0
1180	33.8	645		15.8	7.7
1200	34.3	575		16.5	6.8
1220	33.3	490		17.5	6.0

Fig. 10.15 The effect of laboratory muffle furnace temperature on clay expansion.

expanded LWA particle necessary to give low water absorption, but it is critical at this point that the product needs to be removed and frozen. It can be demonstrated that such an LWA requires to be almost spherical, with a vitrified surface, with an optimum diameter of 14 ± 2mm, with a particle density of 700 ± 100 kg/m^3 and 30-min water absorption of <10%. These developments for the quick assessment and testing of clay have potential for the manufacture of LWA, together with the way it is optimised to prevent the resource being wasted from construction projects and provide sustainable supplies, particularly as occurring in London. Such clays when effectively homogenised and prepared as suitably sized nodules can be fired in a specialist-designed and specialist-adapted kiln for LWA production. The application of additional materials to reduce firing temperatures, without compromising the required properties, will require further research.

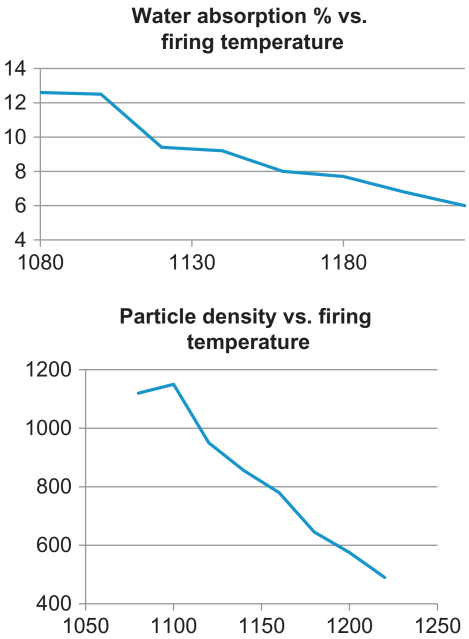


Fig. 10.16 The effect on 'green' clay nodules of furnace temperature from 1080°C to 1220°C on particle density (kg/m³) and 30 min water absorption (%).

10.6 Lightweight aggregate, current kilns and manufacture

By adapting the traditional rotary cement kiln, LWA manufacture developed its own technology. The application of the refractory lined rotating long steel tube shows (Fig. 10.18) the raw feed of fresh nodules being introduced via the feeder at the higher end and passing slowly down the kiln, while injecting the fuel at the other end the hot gases from the burner pass countercurrent to the nodules. As the nodules pass down the kiln, they are dried and heated by the gases and expand when they eventually reach the burning zone. The longer the kiln the better the fuel efficiency, but the more slowly the nodules take to heat. The expansion of the clay results from the liberation of oxygen in the reduction of iron oxides at above 1100°C. Expansion occurs in a relatively short time to the length of time that the clay has been in the kiln (Rudnai, 1963).

Rudnai (1963) also describes the differences between slow and fast heating of suitable clays, observing that maximum expansion occurs between 1110°C and 1200°C after 5 min or less heating time. Expansion is mainly due to the decomposition or reduction of ferrous oxides.

(i) Dissociation of ferric oxide into ferriferous oxide: $6Fe_2O_3 \rightarrow 4Fe_3O_4 + O_2$
(ii) Reduction of ferric oxide to ferrous oxide: $Fe_2O_3 \rightarrow 2FeO + CO$

Clay may also contain some water of crystallisation, it probably being involved in the pyroplastic stage when some of the high-viscosity metal oxides are at melting point.

The current manufacturers of LWA often blend organic matter with the clay. This organic matter is burnt out in the kiln at about 750°C causing voids to reduce the particle density when firing temperatures are below 1150°C. As the organic matter combusts, gas products leave the clay nodule, creating pathways for water to enter the finished particle. This is probably the reason why in 2015 most of the current suppliers of LWA produced high water absorption product, as current demand focuses on LWA for the production of concrete masonry, which requires LWA with high water absorption.

Rudnai (1963) found and advocated a firing time as short as possible for expansion; this was confirmed by research sponsored by Innovate18, a competition sponsored by Crossrail 1 in 2013 for investment awarded to Nustone Ltd. This enabled from the drawing (Fig. 10.17) the construction of a small benchtop rotary kiln and for its operation (Fig. 10.18) to produce an LWA from London clay extracted from the tunnelling operations of Crossrail 1. This kiln (Fig. 10.18) when operated at 1200°C produced in 3 min LWA of the desired expansion and quality. Clay containing between 30% and 25% moisture content at 105°C prepared as nodules (Fig. 10.19), when exposed to 1200°C were transformed into LWA particles (Fig. 10.20) of excellent expansion with a vitrified surface, although the surface would have become increasingly sticky had the particles not been removed quickly from the kiln. This research has also shown that a low 30-min water absorption can be induced that is similar to natural aggregates. Thus when the differences in particle density are taken into account, the 30-min water absorption (eg, 2% with natural aggregate of particle density 2700 kg/m^3) is equivalent

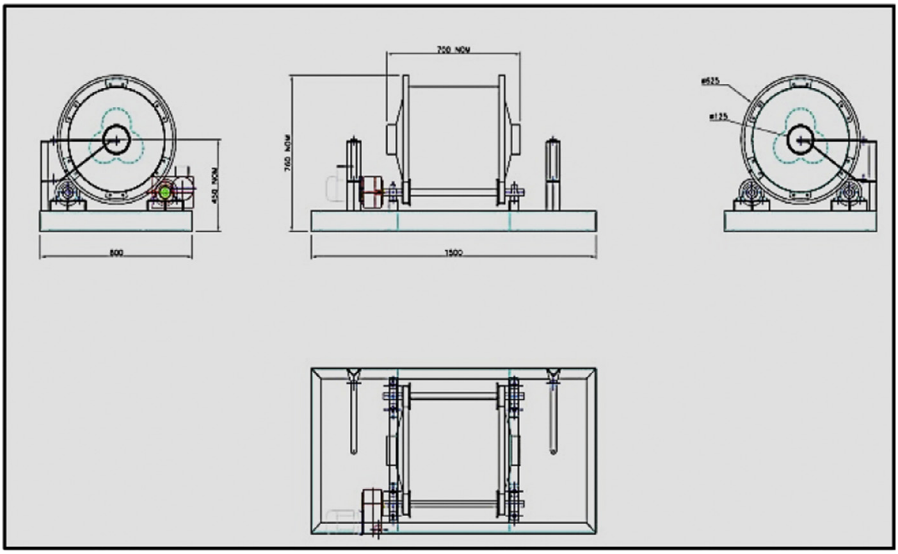


Fig. 10.17 Drawing of the Sherwin bench top kiln.

Fig. 10.18 The Sherwin bench top kiln in operation.

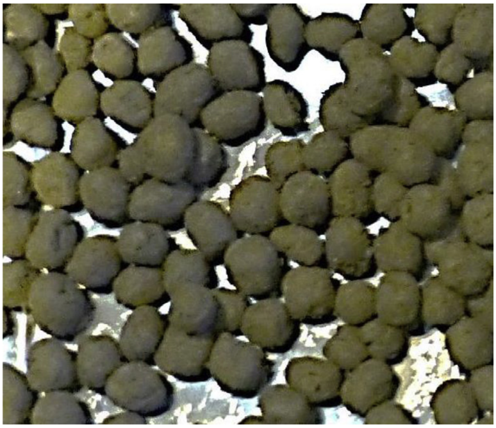


Fig. 10.19 London clay prepared as "green" nodules for firing.

to 7.7% for an LWA of 700 kg/m^3 particle density. Hence, a low water absorption LWA suitable for structural concrete can be achieved.

To increase the rate of heat transfer to this clay, a different type of kiln is required from the traditional long kiln. Research using the benchtop kiln as shown in Fig. 10.18 with a clover leaf cascading alloy lining demonstrated the feasibility of such a kiln and process. As shown in Fig. 10.18, the kiln in operation has a high exhaust gas temperature and intrinsically poor thermal efficiency. To increase efficiency the exhaust heat would need to be recirculated, part to the inlet, part through a heat exchanger to reheat the combustion air, while any remaining heat could be used for other purposes, for example, a waste heat boiler, electricity generator or clay calciner.

Fig. 10.20 LWA produced in the bench top kiln from London clay ex Crossrail. Particle density: 665 kg/m³; 30 min water absorption: 6.8%.

The heat transfer mechanism and production rate for this type of "cascading" kiln will be proportional to the kiln volume. Such a kiln probably within the outer regions of the London area and utilising excavated clay from tunnelling and other civil engineering works, used as the sustainable resource, would be sufficient for the layout of a full-scale plant for:

- Minimum 0.25 million tonne clay stockpile.
- Clay feed into a homogeniser.
- Clay pelletiser forming green nodules.
- A nodule feed to the rotary kiln.
- A fan providing air for the burner.
- The fuel flow and kiln rotation would control the temperature and residence time accurately.
- Part of the exhaust gas from the kiln would pass through a heat exchanger to preheat the air to the fan. The residual heat from the LWA should also be recovered.
- Remaining hot exhaust gas could be used for firing a clay calciner that requires a lower temperature to operate, or the heat would be used to raise steam for electricity.
- The final exhaust would be filtered through a simple bag filter as the exhaust is not very dusty nor does it contain noxious elements.
- Overhead bunker storage for product dispatch, connected to ground stockpile.

Any proposal to situate such an LWA plant in the London area should possibly be at or near the site of a ready mix concrete operation where stockpiles of raw materials are normal for operation, and the LWA aggregate would be ready for immediate use in concrete.

The advantage to the ready mix operation is the production of lightweight concrete giving greater volume to delivery trucks as well as providing concrete that better suits building industry needs. These aspects are completely dependent on the commercial arrangements for such a project.

10.7 Environmental aspects

The embodied carbon dioxide (ECO2) for a m^3 of in situ concrete can be calculated for any mix using either conventional (natural) and/or LW aggregates when the CO_2 produced during manufacture is known for that constituent and before adding the energy necessary for mixing, transporting and placing. ECO2 is based on the total amount of fuel used and the CO_2 produced in kilograms (kg) per metric tonne (te) of product; thus the expression for ECO2 is kg CO_2/te.

While the fuel requirement here to manufacture on a commercial scale the clay-based LWA described is based on the following CO_2 produced:

- The fuel to heat the kiln and materials to 1200°C and
- The heat required to evaporate the moisture and the enthalpy of the exhaust gas.

These are based on the assumption of using natural gas with a net calorific value (CV) of 11,000 Kcal/kg and giving an estimated CO_2 emission of 220 kg/te or 220 kg CO_2/te.

MPA (2014) gives the ECO2 for a "datum" C30 concrete based on Portland cement (CEM I), water, fine aggregates (silica sand) and coarse aggregates (granite). This is presented in Table 10.3 and illustrates how the ECO2 is typically calculated for a "traditional" concrete mix.

The preceding calculation includes the delivery CO_2 for the CEM1 and water, but not the delivery CO_2 for the water contained with the "washed" fine and coarse natural aggregates as given in Table 10.5.

For an equivalent ECO2 Lightweight C30/25 concrete, the concrete density is optimised by the cement combination equivalent to CEM II B-V, using 66.7% CEM I and 33.3% BS EN 450 Class S fly ash[8] together with the coarse aggregate combination where 60% by volume of the natural (granite) aggregate is replaced by LWA. This reduces the density of the finished concrete by 25% as shown in Table 10.4.

Table 10.3 shows that fresh concrete density is reduced by 25% and at the same time reduces the amount of ECO2. The analysis of this data, compared to the datum mix (see Table 10.2) shows a significant reduction in ECO2 resulting from the reduction in the Portland cement (CEM1) content and the application of fly ash. However,

Table 10.3 **Datum for typical calculation of ECO2**

Constituent	Mix kg/m^3 concrete	Embodied CO_2 (kg CO_2/te)	Embodied CO_2 (kg CO_2/m^3 concrete)
Water	190	2	0.4
CEM1	380	930	353.4
Sand	630	8	5.0
5–20-mm granite	1200	25	30.0
Total Fresh Concrete Density	**2400**	Total ECO2	**388**

without the LWA and the extra temperature rise achieved in situ in the hardening process, the activation of the pozzolanic reaction is reduced; hence the benefit of reduced concrete density is lost.

Table 10.5 completes the analysis by necessarily designing a further mix that optimises the fly ash and natural coarse aggregate.

Comparison of the data produced in Table 10.4 shows that there is little significant difference in ECO2 between fly ash concrete using natural aggregates and that of Table 10.3 using fly ash and aggregate combinations on a volume for volume basis, until the benefit of weight reduction of either the concrete element or building is taken into account. In most cases the reduction in mass of 25% with lightweight concrete results in an overall reduction in the weight of the building of at least 12.5%, which in turn allows for options either for reduced foundations or taller and larger buildings for the same foundation. This has a major effect on the amount of steel reinforcement (Rebar), which if reduced will significantly reduce the ECO2, irrespective of the steel

Table 10.4 **Concrete containing fly ash and coarse aggregate combinations**

Constituent	Mix kg per m^3 concrete	Embodied CO_2 (kg CO_2/t)	Embodied CO_2 (kg CO_2/m^3 concrete)
Water	170	2	0.4
CEM1	260	930	241.8
EN 450 fly ash (Class S)	130	150	2.6
Sand	640	8	5.2
5–20-mm granite	300	25	7.5
15-mm LWA	300	220	66.0
Fresh Concrete Density	**1800**		**298**

Table 10.5 **Concrete containing fly ash and natural coarse aggregate**

Constituent	Mix kg/m^3 concrete	Embodied CO_2 (kg CO_2/t)	Embodied CO_2 (kg CO_2/m^3 concrete)
Water	170	2	0.4
CEM1	260	930	241.8
EN 450 fly ash	120	50	6
Sand	650	6	3.9
5–20-mm granite	1200	25	29.6
Fresh Concrete Density	**2400**		**282**

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[8] **Note 8:** MPA Fact Sheet 18 gives an extremely low figure for ECO2 for fly ash. However, it is considered that this should be increased, as while at source the material is a precipitated dry powder, the CO_2 from handling, storage and transfer of this powder to both haulage tankers and into the customer's silos should be taken into account; a higher figure of 2.6 for ECO2 has been assumed, but this did not have a significant impact on the analysis.

source. Imported Rebar to the UK may have been produced by the Basic Oxygen Steelmaking system that produces up to six times more CO_2 than that used by the electric arc furnace favoured by UK producers, who use recycled feedstock (eg, steel recovered from demolished reinforced concrete buildings) (CC, 2015).

The CO_2 data for cement, etc., is given in (MPA, 2015), which also includes the average CO_2 content for delivery.[9] Similarly, the data provided for fly ash includes a delivery component.

The CO_2 from transport as shown in Table 10.5 has also to be taken into account. For this analysis the distances to transport the conventional concrete materials were estimated, and it was also assumed that these products were being used in London, where typical delivery distances are taken as 50 km for the natural aggregates. *NB: The granite coarse aggregate originates from the Scottish Western Highlands, Glensanda Quarry.* The distances and modes of transport for the clay disposal when not used for LWA are unclear, but also requires to take account of the CO_2 arising from the three stages of disposal; (1) transfer from tunnel headings to ground-level storage, (2) transfer from storage by overland transport, either rail or road to river barge to travel to Wallsea Island on the coast of Essex and (3) transfers from the river barge to final point of disposal. This route distance was taken as 100 km, while it was assumed that the transport of clay from the tunnel storage site to the LWA manufacturing plant would be 10 km. The transport of the fresh concrete from the ready mix plant is assumed to be 10 km, and the transport of precast units is taken as 50 km. It is assumed that half the LWA would be used for ready mix operations and half for concrete precast elements.

The CO_2 arising from transport is taken as 63 g CO_2/t/km. Table 10.6 gives typical water contents of fresh clay and the concrete materials being transported and delivered to a ready mix batching plant.

The CO_2 to be embodied for transporting the various materials of concrete/m^3 is given in Table 10.7.

Table 10.6 shows no significant difference in embodied CO_2 based on a cubic metre of concrete by the use of LWA based on Crossrail clay with fly ash in the mix. Compared with standard concrete, the LWA and fly ash mix gives an overall weight saving of 12%. However, where there is also a reduction in the density of the concrete and when the ECO2 is compared on a weight basis (per tonne), very significant reductions are made (Table 10.7). Table 10.8 shows the additional CO_2 embodied (ECO_2) for transport per m^3 of concrete. Table 10.9 shows the effect of concrete density on the weight reduction of a building.

Table 10.6 **Water content: typical % as delivered**

Clay	25
Fine aggregate	8
Natural coarse aggregate	1.5
LWA	0

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[9] **Note 9:** The transport element for the CO_2 emission for delivery of all materials by road has been calculated as 63 g CO_2/t/km, this was based on 1569 g CO_2/km and a 25-te payload (Hillebrand, 2010).

Table 10.7 Embodied CO_2 in the various materials

Material	Assumed distance (km)	Kilograms req'd/m^3	Te CO_2/te
Natural coarse aggregate	50	1218	3.78
Natural fine aggregate	50	690	2.2
Clay disposal	100	206	1.3
Normal-weight (NW) concrete	10	2400	1.5
Lightweight (LW) concrete	10	1800	1.2
Precast elements (NW)	50	2400	7.6
Precast elements (LW)	50	1800	5.7

Table 10.8 Additional CO_2 for transport per m^3 of concrete

Concrete type	Embodied CO_2/m^3	Plus aggregate transport	Plus concrete transport	Plus clay disposal	Total Embodied CO_2
Datum (standard)	332	6.0	4.5	1.3	344
Cement and aggregate combinations	298	2.0	2.4	0	303
Cement combination and natural aggregate	282	6.0	4.5	1.3	293

Table 10.9 Effect of concrete density on the weight reduction of a building

	Embodied CO_2/m^3	Density kg/m^3	Embodied CO_2/t concrete
Standard aggregates	344	2400	826
Combination aggregates	303	1800	560
Reduction %	**12**	**25**	**32**

10.8 CO_2 Savings from the lightweight concrete structures produced

With LWA the density of a standard concrete can be reduced by over 25%, but not all the weight of a structure is concrete; there will be significant quantities of steel, for instance. Assuming the weight of the actual structure is reduced by 20%, this will allow structures to be made larger for the same carbon footprint. Further, they can be built taller for the same foundation size, especially when the foundations are not piled; this is particularly useful for future tunnelling operations in London where the depth of foundations can interfere with planned tunnel routes. Without the specific design and details of the building or structure, the effect on the embodied CO_2 is difficult to quantify, but the following approximation is made on a cubic metre of concrete supplied as in ECO2 from reduced.

- Weight or mass of concrete typically reduced by 20%
- Quantity (tonnes) of concrete used in a building or structure reduced by, say, 15% (smaller foundations, reduced size of structural elements)
- Reduced ECO2 resulting from the structures or works associated with this project 10%

While in reality these estimates are too simple, they are indicative of the process of calculating Embodied CO2. Other prime benefits of LWA concrete containing fly ash need to be taken into account, such as reduced permeability, increased crack control, faster strength gain at higher temperatures and greater final in situ strength. This supports most of the properties detailed in the publications for lightweight structural concrete (C&CA, 1970), etc.

10.9 Embodied CO2 Summary

Concrete is produced and sold by the cubic metre. As a constituent of concrete, natural aggregates are purchased by the tonne; therefore the particle density of coarse natural aggregate is critical to Embodied CO2 and the weight of concrete.

Coarse aggregate particle density (PD) has the greatest effect on concrete density. Hence as the PD of:

Natural aggregate (granite or limestone) is about 2750 kg/m^3, this is equivalent to 1.2 te/m^3; and
LWA is about 700 kg/m^3; this is equivalent to 0.31 te/m^3.

Thus **1.0 te of LWA is equivalent to 3.9 te of natural aggregate.**

This is most important when considering the cost of LWA.

While there is no significant reduction in embodied CO_2/m^3 of concrete, using either aggregate combinations (that include LWA) or natural aggregates, but with the inclusion of fly ash (the addition of fly ash reduces both ECO2 and improves durability) and LWA, the standard 20°C water stored cube strength is reduced, therefore requiring more CEM I to compensate. However, when the evaluation is made by comparing the embodied CO_2/te of concrete, then a 25% reduction results. Further, when the temperature during hydration is used more effectively, a greater percentage of fly ash can be used, for example, 40–50%.

Translating this into the effects of building structures in London, there is almost 10% reduction in embodied CO_2 when using LWA in combination with natural aggregates. This analysis does demonstrate that the transport component of the calculated ECO2 resulting from the use of London clay as a raw material for LWA is minor. However, the opportunity to include LWA does reduce weight and thus total binder content does have a major influence in the final analysis.

10.10 Conclusions

This chapter has focused on the density of structural/reinforced concrete and how it can be adjusted to give advantage to the national economy of future construction. The production of such concrete, however, depends not only on the availability, but also on the sustainability of an appropriate LWA. This chapter has advocated:

- Reducing the cost of clay waste disposal from tunnelling and other deep construction;
- Producing an LWA from that clay to give flexibility to the specification of concrete density;
- Concrete with reduced ECO2;
- Increased durability through reduced permeability and chemical stability;
- The control of cracking;
- Smaller foundations or taller structures; and
- A prospective on concrete properties, especially strength.

Lower compressive strength of concrete is inevitable with reducing density. However, by more efficiently applying the energy of the greater binder content, the in situ strength can be optimised to suit the structural application; foundations do not require high-strength concrete, but reliability and stability. By offsetting the long-term benefits of lightweight concrete, the erroneous results often produced from standard cubes cured at 20°C under water can be avoided in situ. Alternatively, permitting or even encouraging the temperature of in situ concrete to actually rise and to apply these elevated temperatures to achieve maturity earlier, combined with the concrete of appropriate mix design, can activate cement combinations with greater proportion of compatible materials, that in the long term provide greater durability. Mix design can vary the density by using combination aggregates that allow the strength/density ratios to be optimised for any structure. However, to fully utilise this opportunity, standard strength testing needs to be supplemented with in situ thermodynamics required to replicate in situ concrete performance, and thus curing in insulated pods is recommended.

Recommendations

- Clays from excavations should not be wasted, but should be assessed either as a resource for LWA manufacture or as a ground calcined alumina-silica for application with Portland cement as binder.
- The adoption and application of aggregate combinations enables the specification of structural concrete density between 2400 and 1600 kg/m^3.
- The specification of structural lightweight concrete allows for greater construction efficiency and for reducing the carbon footprint of concrete.
- The adoption of test procedures that more closely represent the in situ performance of concrete.

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“While the chapter was in the final stage of production, Philip Owens passed away at the age of 85. Philip’s ambitions were to manufacture lightweight aggregate from waste clay and it is hoped that this chapter serves that purpose.”

Sustainability of masonry in construction

11

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11.1 Introduction

This chapter provides an overview of masonry as a sustainable construction material. It deals primarily with clay brickwork and concrete blockwork, these being the main types of masonry used in the UK. The chapter describes their manufacture and principal properties with regard to sustainability. The historical use of masonry is briefly reviewed, and recent changes in the use and forms of masonry construction are highlighted.

The broader issues associated with sustainability are discussed and current methods for quantifying the whole life environmental performance of masonry walling used in residential and commercial buildings within the UK are examined. Examples of masonry buildings that are ‘more sustainable’ in terms of their operational energy and/or material usage are given, together with a brief outline of possible future developments in the masonry area.

11.2 Additional sources of information

In view of the limitations on size, much detail has necessarily had to be left out of this chapter. For additional background information on the engineering properties of brick and block masonry, the reader should consult books such as *Civil Engineering Materials*, 5th Edition (Jackson and Dhir, 1997), *ICE manual of construction materials* (Forde, 2009) and *Construction Materials: Their Nature and Behaviour* (Illston and Domone, 2001). Information is also available from the websites of the UK manufacturers of masonry products and the various trade associations within the UK masonry industry. The latter include:

- Aircrete Products Association;
- Brick Development Association;
- British Lime Association;
- British Precast Concrete Federation;
- Concrete Block Association;
- Hemplime Construction Products Association;
- Mortar Industry Association;
- Quarry Products Association;
- Stone Federation Great Britain;
- Traditional Housing Bureau.

Sustainability of Construction Materials. http://dx.doi.org/10.1016/B978-0-08-100370-1.00011-1

The British Masonry Society also publishes papers on all aspects of masonry construction and performance.

The history of brick masonry and brick manufacturing in the UK, and the different mortars used in brick and stone masonry, are comprehensively described by Lynch (1994). Other useful sources of information include the *Chilterns Buildings Design Guide: Chilterns Brick Supplementary Technical Note* (Chilterns Conservation Board, 2005) as well as the Internet generally. Pevsner Architectural Guides and the Buildings Books Trust also give numerous examples of historically significant brick and stone masonry buildings throughout Great Britain.

Further details of the environmental performance of UK construction materials and forms of construction, including masonry, are available from the Building Research Establishment (BRE) website (http://www.bre.co.uk) and their associated publications and software packages. Useful introductions to life-cycle assessment (LCA), the generic process that underpins environmental sustainability, are available from the SimaPro 7 webpage of PRé Consultants (SimaPro7, 2007) and the ISO 14040 series of voluntary standards dealing with LCA (International Organization for Standardization, 2006).

11.3 Definitions

Masonry is the generic term used to describe an assemblage of preformed ‘units’ laid in a bed of mortar. In the UK a range of units are available, including clay bricks (both fired and unfired), concrete bricks, dense and lightweight aggregate concrete blocks, aerated concrete blocks, and natural and reconstituted stone. Although previously available, calcium silicate (sand-lime) bricks are no longer marketed in the UK and are not considered here.

Mortar is essentially a mixture of fine sand, water and a binding agent. Hydrated lime or a plasticising agent (either in powdered or liquid form) is often added to improve the physical properties of the mortar in its fresh or hardened states. A colouring agent may also be included to improve the overall appearance of the masonry. Essentially, mortar bonds the individual units together and provides a degree of weatherproofing to the structure being built. In modern forms of masonry construction, the binding agent in mortar is normally based on Portland cement (CEM I), or more commonly one of the CEM II blended cements, whereas in older masonry structures some form of lime would have been used, possibly with added ash or soot as a colorant.

In the UK, bricks have traditionally been defined in terms of unit size. Since 1969 this has been standardized at 215 mm long × 102.5 mm wide (thick) × 65 mm high, although the new European standards for masonry no longer limit the size or shape of bricks or blocks; instead, bricks or blocks are now both referred to as masonry units. The UK tradition of differentiating blocks and bricks by face size is, however, likely to continue (British Standards Institution, 2011b). Typical types of bricks used in the UK are shown in Fig. 11.1A–C.

Blocks may be solid, cellular or hollow (Fig. 11.2). They are produced in a number of face sizes from 390 mm long × 190 mm high up to 610 mm long × 270 mm high, in the case of aerated blocks. The thickness of blocks ranges from 75 to 300 mm.

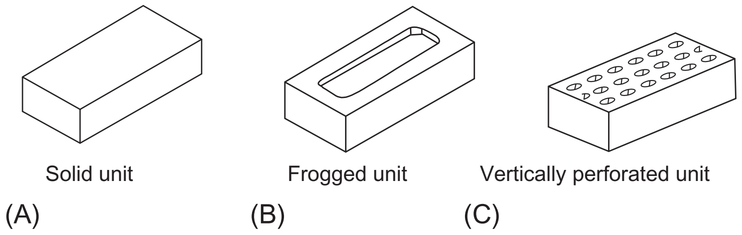


Fig. 11.1 Common types of UK brick.

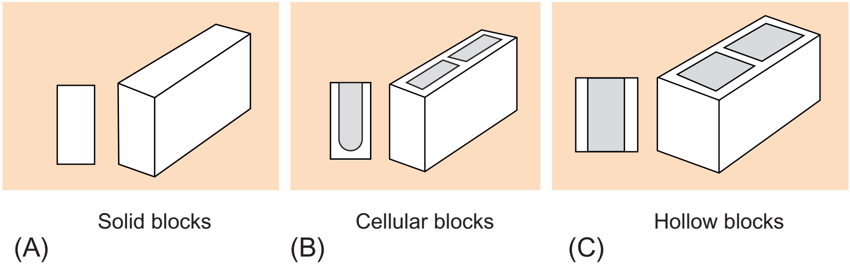


Fig. 11.2 Common types of UK block.

Reconstituted and natural stone products are produced in various sizes depending on the finish required, for example, random or coursed. The thickness of the mortar joint for masonry construction has now been standardized at 10 mm apart from thin-joint blockwork, where it is 3 mm.

11.4 Facts and figures

Approximately 1.8 billion fired-clay bricks are produced annually in the UK (2014). This accounts for 95% of all those sold, with the remaining 5% being concrete bricks. The current market value of clay bricks is about £550 million per annum with some 1200 varieties of clay brick available (Brick Development Association, 2007). Facing bricks, which account for over 90% of UK sales, are used primarily in the domestic housing market. Engineering bricks, with their higher strengths and lower porosity, are used for load-bearing masonry structures and in more aggressive environments generally.

The UK brick industry consumes 4.5 million tonnes of clay each year and is energy intensive, its total annual energy consumption being around 706 kWh/tonne of primary energy (Carbon Trust, 2010). This is, however, less than 1.5% of the total energy consumed by the UK manufacturing industry.

Concrete block sales are reported in terms of square metres of blockwork; in the UK this is currently around 57 million m^2 per annum (Department for Business, Innovation and Skills, 2015). Aggregate concrete blocks account for around two-thirds of sales, the remainder being aerated blocks. No precise figures appear to exist regarding the primary energy consumed in block manufacturing or stone production. In relation to the latter it should be noted, however, that approximately 30% of the natural stone used in the UK is now imported from countries as far away as India and Brazil (personal communication with the Stone Federation of Great Britain, 2007).

11.5 Manufacture of masonry units and mortar

11.5.1 Fired-clay bricks

Three basic techniques are used for the initial shaping of fired-clay bricks (Hammett, 2004).

1. *The soft-mud process.* The clay is thrown into moulds either by hand or by machine. The bricks are then removed from the moulds, dried and fired. Bricks made by this process contain 25–30% water and are known as ‘stock bricks’.
2. *The extruded, wire-cut process*. The clay is mixed with approximately 13–18% water to a stiff plastic consistency and forced through a die with dimensions slightly greater than the length and width of the brick. Taut wires then cut this continuous ‘sausage’ of clay into individual bricks. This is the principal method of brick forming and accounts for approximately 65% of UK production.
3. *Semi-dry pressing.* This method uses hydraulic presses to push semi-dry clay into steel moulds and is reserved for Fletton bricks produced in the Peterborough area.

After shaping, the ‘green’ bricks are allowed to dry for up to 5 days and then fired at temperatures of between 900 and 1150°C, depending on the type of clay. The chemical and physical changes occurring during the firing process are complex and involve the breakdown of the original clay minerals and the formation of new crystalline materials and glass phases (British Geological Survey, 2007). These give the brick its mechanical strength and durability. The firing process and subsequent cooling of the bricks takes between 40 and 150 h, depending on the type of clay and the kiln used. Four types of kiln are available: intermittent, clamp, continuous multichamber and continuous tunnel. In practice, the tunnel kiln is used in most large-scale works as it is more efficient, with a complete firing cycle typically taking between 2 and 3 days.

For many years recycled waste materials from other industries have been utilized in the manufacture of clay bricks. These include pulverized fuel ash (PFA), blast furnace slag, coke breeze and coal slurry, ground glass (cullet), sewage sludge, paper ash, bone ash and sawdust (Smith, 2012), accounting for around 10% of raw materials used. Some of these are used to improve or modify specific properties of the brick, for example, its frost resistance or colour, while others that are combustible may simply act as a source of fuel, reducing the amount of external energy required during the firing stage. Waste materials such as ground glass also act as fluxing agents, reducing firing temperatures and times, as well as emissions to air. Recent research has shown that when ground glass is used in brick manufacturing, energy savings of up to 20% are possible (WRAP, 2006).

In practice, the greater use of recycled waste materials in brick manufacturing is currently somewhat limited by handling, supply and quality issues, as often only small quantities of these materials, which potentially may be hazardous, are available. Nevertheless, the ability of the clay brick industry to utilize waste products that would otherwise go to landfill is extremely beneficial from a sustainability viewpoint.

11.5.2 Unfired-clay bricks and blocks

Although available in Europe for a number of years, and used widely throughout the world for low-rise construction, unfired bricks and blocks (also known as adobe blocks, rammed earth blocks) have only recently been introduced to the UK and are still at the developmental stage. Essentially, they consist of raw clay mixed with sand and sawdust which, after shaping, is artificially air-dried for approximately 2 days and then used in construction. As they are not kiln fired, their embodied energy (the quantity of energy required to manufacture a material or product) is very low. For example, the unfired brickwork in a test house had embodied energy of 146 kWh/tonne and embodied carbon of 44.6 kg CO_2/tonne. This is about 14% of the value for comparable fired-clay brickwork and 24% of that for lightweight blockwork (Morton, 2006). Unfired bricks are normally used for internal non-load-bearing walls and are laid in weak clay, rather than cement-based, mortars (Errol Brick, 2007), and rendered or plastered with either clay or lime-based materials.

11.5.3 Concrete blocks

Two basic types of blocks are available in the UK: aggregate and autoclaved aerated concrete blocks (Aircrete).

11.5.3.1 Aggregate blocks

Two types of aggregate block are produced: dense and lightweight. Both consist of a binder, which is normally a form of Portland cement (CEM I) or a blended Portland cement (CEM II), together with water and graded aggregates of 10–14 mm maximum size. In addition, PFA may sometimes be used as a partial cement replacement. For lightweight blocks, furnace bottom ash and some type of lightweight aggregate such as Leca or Lytag are used, whereas natural aggregates are used for dense blocks. The manufacture of both dense and lightweight blocks involves the compaction of a very dry concrete mix into individual moulds, after which the ‘green’ blocks are immediately pushed out onto a pallet and taken away for curing.

11.5.3.2 Autoclaved aerated concrete blocks (Aircrete)

The materials used in the manufacture of Aircrete blocks are PFA, sand, cement, lime and water. The PFA, sand and water are initially mixed to form a slurry which is heated and mixed with cement and lime. A small quantity of aluminium powder is then evenly dispersed through the mixture before it is poured into moulds.

The aluminium initiates a chemical reaction, generating minute bubbles which form the characteristic Aircrete structure. When the mixture has partially set, the resultant ‘cakes’ are wire-cut into blocks of predetermined size, and transferred to autoclaves for high-pressure steam curing. During this process the ingredients combine to form the calcium silicate hydrates that provide the mechanical strength of the finished blocks.

Aircrete production is environmentally friendly when compared with many other construction materials as a large proportion of the block is PFA, a product that would otherwise be used in landfill. The process is also highly efficient with most waste material and energy being recycled back into the production process itself (H+H Celcon, 2007).

11.5.4 Mortar

The sand usually used in masonry mortars is ‘fine sand’, the grain size being smaller than that of the ‘coarse sand’ used for the manufacture of concrete. In addition, in masonry mortars the shape of the sand grains is essentially rounded or ‘soft’. This is in contrast to the sand used in concrete, where the individual grains are more angular, or ‘sharp’. A range of mortar mixes is available for use in masonry construction (Table 11.1). Traditionally these have been specified in volume terms, with sufficient water being added to the mix to achieve the desired workability. Weak mortars are more able to accommodate brick or block movement whereas high-strength mortars will provide a better bond, higher lateral walls strengths and increased frost resistance. Mortar mix selection depends on the types of units being laid and the degree of exposure, with higher strength mixes required in more exposed situations generally (see Table 11.4).

Although once common practice, less than 25% of the mortar used in the British market each year is now mixed on site using traditional cement mixers (Beningfield, 2002). Instead, most mortars are weigh-batched in factories under controlled conditions and then delivered to site. Two main types of factory-produced mixes are

Table 11.1 **Mortar compressive strength classes, composition and designation**

Compressive strength class[a]	Prescribed mortars (proportion of materials by volume)		Traditional mortar designation
	Cement:lime:sand with or without air entrainment	Cement:sand with or without air entrainment	
M12	1:0 to ¼:3	1:3	(i)
M6	1:½:4 to 4½	1:3 to 4	(ii)
M4	1:1:5 to 6	1:5 to 6	(iii)
M2	1:2:8 to 9	1:7 to 8	(iv)

[a] The number following the M is the compressive strength for the class at 28 days (in N/mm^2).

Adapted from UK National Annex to BS EN 1996-1-1:2005 + A1:2012 (British Standards Institution, 2005. BSEN 1996-1-1:2005 + A1 2012. Eurocode 6. Design of Masonry Structures. General Rules for Reinforced and Unreinforced Masonry Structures. British Standards Institution, London.)

available: lime–sand mixes to which cement and water is added on site to produce a masonry mortar, and ready-to-use mortar.

Ready-to-use mortars have guaranteed mix proportions and overcome any potential problems relating to site mixing. Two types of ready-to-use mortar are available: wet and dry. Wet ready-to-use mortars incorporate a retarding agent and are stored in tubs on site. They require no further mixing and are fully usable for a specific period, typically 36 h. Dry ready-to-use mortars are stored in silos or bags. The silos are delivered to site complete with integral mixers and require only power and water supplies to be connected in order to mix the mortar (Mortar Industry Association, 2005).

The mixes shown in Table 11.1 contain Portland cement and should not be confused with traditional lime mortars. These are occasionally specified for new masonry construction and use hydraulic lime, rather than cement, as the binding agent. Traditional lime mortars are considered to be more environmentally friendly than cement-based mortars, although they may be slower to set and gain strength. The stability of modern forms of thin masonry walling built with traditional lime mortars may therefore be an issue during the construction phase, and care should be taken with their use. Further information on traditional lime mortars is available from both the UK Brick Development Association Ltd (Brick Development Association, 2001a) and Lime Technology (Limetechnology, 2007).

Mortars for use in thin-joint blockwork are normally cement–sand mixes that contain additional polymer reinforcement as well as specialist shrinkage or plasticising agents to produce a workable mortar at low water:cement ratios (Building Research Establishment, 1998).

11.6 Standards for masonry

As part of the move towards a single European market, new European Standards for construction materials and products, including masonry, are being introduced. Where already adopted, these have the status of a British Standard and are referred to as a BS EN. In addition, a new series of European structural design codes (Eurocodes) is gradually being introduced for all materials. Table 11.2 shows some of the new standards for masonry. It should be noted that the manner in which masonry products are specified and tested in these new standards may be different from previous British Standards. A useful summary of the main differences for clay bricks and concrete blocks is provided by Ibstock (2003) and the Concrete Block Association (2006), respectively.

11.7 Properties of masonry

This section briefly describes some of the principal properties of masonry and masonry units. For further details the reader should consult the relevant standard(s) listed in Table 11.2.

Table 11.2 European standards for masonry

	European standard
Masonry units	
Clay masonry units	BS EN 771-1:2011
Calcium silicate masonry units	BS EN 771-2:2011
Aggregates concrete masonry units (dense and lightweight aggregates)	BS EN 771-3:2011
Autoclaved aerated concrete masonry units	BS EN 771-4:2011
Manufactured stone masonry units	BS EN 771-5:2011
Natural stone masonry units	BS EN 771-6:2011
Mortar	
Rendering and plastering mortar	BS EN 998-1:2010
Masonry mortar	BS EN 998-2:2010
Design of masonry structures	
Design of masonry structures. General rules for reinforced and unreinforced masonry structures	BS EN 1996-1-1:2012
Design of masonry structures. General rules. Structural fire design	BS EN 1996-1-2:2005

11.7.1 Compressive strength

The strength of masonry units can vary considerably. Typical values are shown here (Institution of Structural Engineers, 1996):

- clay bricks, 15–150 N/mm^2;
- lightweight aggregate blocks, 2.9–25 N/mm^2;
- dense aggregate blocks, 2.9–40 N/mm^2;
- aerated blocks, 2.9–7.3 N/mm^2;
- natural stone, 16–250 N/mm^2.

In practice, the strength of a brick, block or stone masonry wall is considerably lower than the compressive strength of the individual masonry units from which it is made. This is due to the presence of the weaker mortar joints in which the units are laid. In addition, the load-bearing capacity of masonry walls decreases with increasing slenderness, that is, the ratio of (effective) wall height to (effective) wall thickness (Curtin et al., 2006).

11.7.2 Density

Two types of fired-clay masonry units are specified in BS EN 771-1 (British Standards Institution, 2011a), namely, low density (LD) for use in protected masonry (gross dry density ≤1000 kg/m^3) and high density (HD), although in 2015 the revision of the standard, currently under review, will refer to Protected (P) and Unprotected (U) masonry units, depending on the intended use. The majority of UK bricks are classified as high density (unprotected); owing to the presence of perforations and frogs, the weight of UK bricks may vary from around 1.7 to 3.6 kg. The net dry density of aggregate concrete blocks (ie, allowing for the volume of voids) ranges from 650 to 2400 kg/m^3 with aerated blocks normally within the range 300–1000 kg/m^3.

11.7.3 Configuration

Masonry units in BS EN 1996-1-1 (British Standards Institution, 2012) are grouped in one of four specific categories depending on factors such as volume of voids present and direction of the voids and horizontal or vertical. In the case of aggregate concrete blocks, for example, the groups are

- Group 1: <25% formed voids;
- Group 2: >25% and <60% formed vertical voids;
- Group 3: >25% and <70% formed vertical voids;
- Group 4: >25% and <50% formed horizontal voids.

Most aggregate concrete block units manufactured in the UK fall within Groups 1 and 2.

11.7.4 Movements in masonry

Movement in masonry is considered here under the following headings: elastic movement, moisture movement, thermal movement and creep.

11.7.4.1 Elastic movement

The movement of brick and block masonry under short-term load is a function of the movement of the individual bricks, blocks and mortar that together form the masonry. As such, it can vary considerably depending on the type of unit and mortar used. For example, the elastic modulus of clay bricks ranges from 3.5 to 35 kN/mm^2 while that of mortar can vary between 0.6 and 25 kN/mm^2 (Jackson and Dhir, 1997). The elastic modulus of brick or block masonry is consequently also very variable, typically being in the range 1–30 kN/mm^2.

11.7.4.2 Moisture movement

Clay bricks expand or contract with increases or decreases in moisture content. In addition, they undergo long-term irreversible expansion due to adsorption of water vapour from the atmosphere. The rate of irreversible expansion is initially high but decreases with age. Concrete blocks, stone and mortar expand or contract with changes in moisture content but, unlike clay bricks, they undergo long-term drying shrinkage.

Moisture movement of brick, block or stone masonry is therefore the composite effect of short-term changes in moisture content and irreversible expansion/drying shrinkage of the masonry units and mortar in the joints. The net effect is generally considered to be a gradual expansion in most types of clay brickwork (although an initial shrinkage may occur) whereas concrete blockwork and stone masonry contract with time.

The age of the units at the time of laying influences the amount of movement in the masonry—the longer bricks/blocks are left to stand, the smaller the ultimate moisture expansion/drying shrinkage of the brickwork and blockwork, respectively.

In order to avoid in-service problems, the use of kiln-fresh clay bricks for new masonry construction should, in particular, be avoided.

11.7.4.3 Thermal movement

Masonry units and mortar expand or contract in response to short-term changes in ambient temperature. In the case of clay brickwork, movement in the vertical direction may be 50% higher than in the horizontal direction due to the greater proportion of mortar, which has a higher coefficient of expansion than brick.

In order to accommodate thermal and moisture movements, soft vertical movement joints are usually incorporated in masonry walls at regular intervals: typically every 12–15 m in the case of clay brickwork, and every 6–9 m for blockwork.

11.7.4.4 Creep

When subject to a sustained load, masonry undergoes an initial elastic deformation followed by a long-term, time-dependent, deformation. The latter consists of creep together with drying shrinkage or moisture expansion, according to the type of masonry unit. Creep is important as it relieves stresses in restrained masonry and reduces the tendency for tensile shrinkage cracks to develop in restrained blockwork panels, for example.

Table 11.3 shows the ranges of values for long-term moisture movement, coefficients of thermal expansion and creep coefficients for different types of masonry.

Table 11.3 **Ranges of coefficients of creep, moisture expansion or shrinkage, and thermal properties of masonry**

Type of masonry unit	Final creep coefficient[a] Φ_∞	Long-term moisture expansion or shrinkage[b]	Coefficient of thermal expansion, α_t (10^{-6}/K)
Clay	0,5 to 1,5 (1,5)	−0,2 to +1,0 (0,5)	4–8 (6)
Dense aggregate concrete and manufactured stone	1,0 to 2,0 (1,5)	−0,6 to −0,1 (−0,2)	6–12 (10)
Lightweight aggregate concrete	1,0 to 3,0 (1,5)	−1,0 to −0,2 (−0,4)	6–12 (10)
Autoclaved aerated concrete	0,5 to 1,5 (1,5)	−0,4 to +0,2 (−0,2)	7–9 (10)
Natural stone			
Magmatic			5–9 (10)
Sedimentary	[c]	–0,4 to +0,7 (0,1)	2–7 (10)
Metamorphic			1–18 (10)

---

[a] The final creep coefficient $\Phi_\infty = \varepsilon_{c_\infty} / \varepsilon_{\mathrm{el}}$, where ε_{c_∞} is the final creep strain and $\varepsilon_{\mathrm{el}} = \sigma / E$.

[b] Where the long-term value of moisture expansion or shrinkage is shown as a negative number, this indicates shortening; a positive number indicates expansion.

[c] These values are normally very low.

Values shown in parentheses are the values adopted in UK National Annex to BS EN 1996-1-1:2005.

Adapted from BS EN 1996-1-1:2005 + A1:2012 (British Standards Institution, 2012).

(The reader should note the use of a comma rather than a full stop to represent the decimal point, this being the European convention.)

11.7.5 Durability

The durability of masonry products is normally expressed in terms of their resistance to freeze/thaw and, in the case of clay bricks, their active soluble salt content.

11.7.5.1 Freeze/thaw resistance

1. *Clay bricks*. BS EN 771-1 specifies that bricks should comply with one of three categories according to their degree of exposure to saturation and freezing: frost resistant (F2), moderately frost resistant (F1), not frost resistant (F0).
2. *Concrete blocks*. For extreme exposure conditions and frost attack, aggregate blocks with a strength in excess of 7.3 N/mm^2 or Aircrete blocks may be used.
3. *Natural stone*. Within the UK, physical damage due to frost is considered to be relatively rare except in conditions of extreme exposure (Institution of Structural Engineers, 1996); however, where required natural stone masonry units can be tested in accordance with EN 12371 and a declaration of the number of freeze–thaw cycles before damage occurs is made by the supplier.

11.7.5.2 Category of active soluble salts

Fired-clay bricks, in particular, may contain water-soluble salts (especially those of sodium, potassium and magnesium) that can attack the mortar, or migrate to the surface of the brickwork causing efflorescence. BS EN 771-1 specifies that bricks should comply with one of three categories according to the degree of active water soluble salts: S0, S1 and S2. These limits ensure that, under the particular service conditions, damage will not occur to the masonry units, mortar or render (if any).

Table 11.4, adapted from BS 5628: Part 3 (British Standards Institution, 2005a), shows the quality of masonry units and mortar designations necessary to achieve durable masonry construction for specific exposure conditions.

11.7.6 Water absorption

Water absorption is important in relation to whether clay brickwork in particular should act as a *raincoat* or an *overcoat* on buildings. Bricks with high absorption, for example, may act as an overcoat, soaking up large amounts of rain which is then slowly released back to the atmosphere. Low absorption bricks, on the other hand, tend to shed rainwater. This can lead to unsightly streak marks down the face of the brickwork as well as water penetrating the brickwork through incompletely filled mortar joints. A low water absorption figure is specified for engineering bricks (Table 11.5), which are frost resistant, and for bricks used as dampproof courses although water absorption, in itself, is not a reliable indicator of a brick's resistance to frost attack (British Standards Institution, 2005a).

Table 11.4 Quality of units and mortar for durable masonry

	Quality of masonry units and appropriate mortar designations			
Masonry condition or situation	**Clay units**	**Aggregate concrete bricks**	**Aggregate concrete blocks**	**Remarks**
(A) Work below or near external ground level				
A1. Low risk of saturation		Without or with freezing	Without or with freezing	
Without freezing	LD—F0 and S0	Compressive strength 16.5 N/mm² or above in (iii)	(a) of net density ≥1500 kg/m³	Some types of autoclaved aerated concrete block may not be suitable. The manufacturer should be consulted
	or HD—F0, F1 or F2 and S0, S1 or S2 in (i), (ii) or (iii)		or (b) Made with dense aggregate conforming to BS EN 12620 or (c) Having a compressive strength of 7.3 N/mm²	
With freezing	HD—F1 or F2 and S0, S1 or S2 in (i), (ii) or (iii)		or	In sulphate-bearing ground

			(d) Most types of auto-claved aerated block (see remarks) All in (iii) or (iv) (see remarks)	conditions, the recommendations in 5.6.4 should be followed. Where designation (iv) mortar is used, it is essential to ensure that all masonry units, mortar and masonry under construction are protected fully from saturation and freezing (see A.4.1.3.2 and A.5.1.1)
(E) Internal walls and inner leaves of cavity walls above DPC level				
Internal walls and inner leaves of cavity walls	LD—F0 and S0 or HD—F0, F1 or F2 and S0, S1 or S2 in (i), (ii), (iii) or (iv) (see remarks)	Compressive strength 7.3 N/mm^2 or above in (iii) or (iv) (see remarks)	Any in (iii) or (iv) (see remarks)	Where designation (iv) mortar is used, it is essential to ensure that all masonry units, mortar and masonry under construction are protected fully from saturation and freezing (see A.4.1.3.2 and A.5.1.1)

LD = low density; HD = high density.
(i) Designation (i) mortar (M12).
(ii) Designation (ii) mortar (M6).
(iii) Designation (iii) mortar (M4).
(iv) Designation (iv) mortar (M2).
Adapted from BS 5628: Part 3(2005) (British Standards Institution, 2005a. BS 5628-3:2005. BS 5628-3:2005. Code of Practice for the Use of Masonry. Materials and Components, Design and Workmanship. British Standards Institution, London).

Table 11.5 **Specification for UK engineering and DPC bricks (BS EN 771-1:2011 (British Standards Institution, 2011a))**

Performance characteristic	Clay engineering bricks	
	Class A	Class B
Compressive strength (N/mm^2)	125	75
Water absorption (% by mass) and also when used as DPC units	4,5 (and DPC1)	7,0 (and DPC2)
Freeze/thaw resistance category	F2	F2
Active soluble salts category	S2	S2

DPC1 bricks are required for buildings. DPC1 or 2 may be used in external works.

11.7.7 Fire resistance

Most masonry products are inherently resistant to fire, as they contain a mass or volume fraction of ≤1.0% of homogeneously distributed organic materials. They are categorized as Class A1 in terms of BS EN 13501-1:2007 (British Standards Institution, 2007); that is, they will not contribute in any stage of the fire. Masonry walls consequently perform well in fire with, for example, 100-mm solid clay brickwork walls having 120 min fire resistance (British Standards Institution, 2005b).

11.8 Historical use of masonry

Stone masonry, as a form of construction, has existed since the beginnings of civilization. The ancient pyramids in Egypt were built during the second and third millennia BC using stone laid in gypsum mortar. Sun-dried bricks first appeared in southern Turkey c.8000 BC while fired-clay bricks were used in the third millennium BC (Lynch, 1994). Fig. 11.3 shows brick and stone masonry from the second century AD.

The Romans introduced fired bricks to England although, after the fall of the Roman Empire, the use of masonry effectively ceased. Stone masonry construction began again in Europe during the 9th and 10th centuries and thousands of churches and cathedrals were built in masonry during the Middle Ages. Bricks were reintroduced to the UK from the Netherlands in the 12th and 13th centuries, and UK brick manufacturing recommenced at this time. Examples of brickwork from this period still exist (Fig. 11.4).

The demand for bricks increased rapidly following the Great Fire of London in 1666 and again during the Industrial Revolution in the 19th century when brick became the dominant construction material with an annual production in excess of 9 billion units. The move to a unified metric size for UK bricks took place in 1969, although some manufacturers still produce imperial-size bricks for renovation work and extensions to existing buildings.

The market for masonry, and the manner in which it is used, has changed greatly during the last hundred or so years. This is due to the introduction of materials such as

Fig. 11.3 The Roman Library of Celsus at Ephesus.

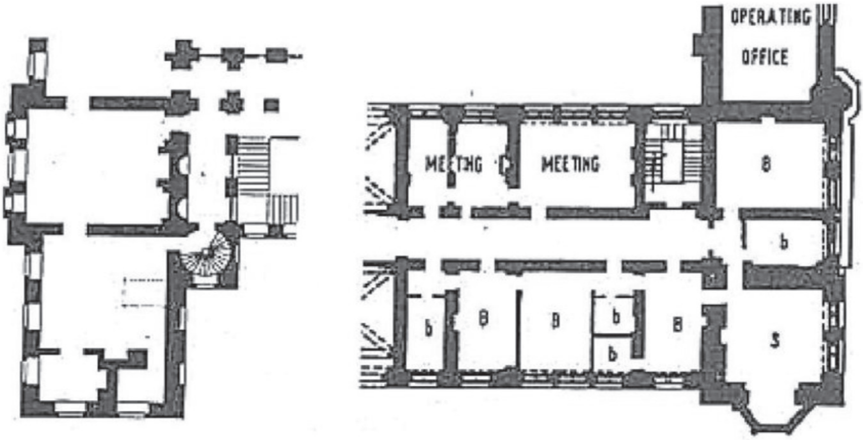
Fig. 11.4 The north side of the chancel at Holy Trinity Church, Hull, UK.

reinforced concrete and structural steel which, unlike masonry, are able to resist tensile bending stresses. This has allowed new types of bridges, for example, to be built and has led to an overall decline in the use of masonry for traditional civil engineering structures such as arches, retaining walls and sewers.

The development of steel and concrete structural frames during the early part of the 20th century has also meant that clay brick masonry no longer performs its traditional function as a load-bearing material in buildings (Sutherland, 1993).

Instead, its function is now essentially limited to that of a decorative cladding, insulating and weatherproofing the interior.

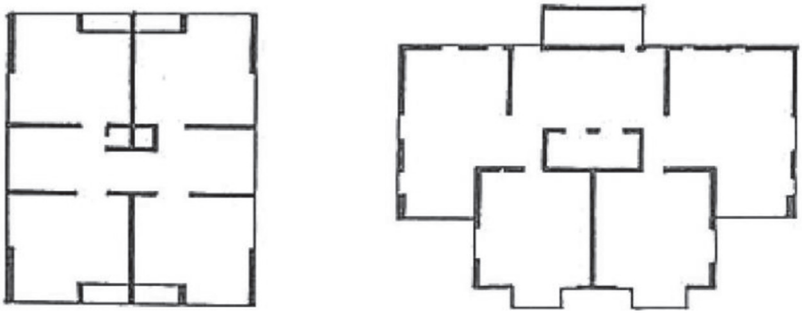
The introduction of structural design codes for masonry, in particular, has enabled the thickness of walling to be greatly reduced during the last century (Fig. 11.5), and



Kings Weston 1712–4

St Pancras Hotel 1867–73

(A)



Typical layouts of multi-storey load-bearing buildings of the 1960s

(B)

Fig. 11.5 Comparison of wall thicknesses in (A) 18th, 19th and (B) 20th centuries in relation to overall areas of buildings (Sutherland, 1993) (RJM Sutherland, Proceedings (5) International Masonry Conference, British Masonry Society, May 1993. Copyright International Masonry Society). Each plan is approximately to scale within itself, but the plans are not all reproduced to exactly the same scale.

cavity wall construction, with its unloaded outer leaf, has become the norm for masonry buildings in the UK and other European countries with similar, that is, wet, climates. In addition, traditional lime mortars have been replaced by quick-setting cement-based mortars which, although usually stronger, are more prone to cracking generally. Efforts to promote the increased use of lime mortars in masonry, with their greater ability to accommodate movements, are nevertheless underway (Sumacon, 2007).

A particular feature of masonry construction over the past 60 or so years has been the introduction of concrete blocks. These are more thermally efficient than clay bricks and, due to their size, enable increased rates of production to be achieved on site. The introduction of higher standards of thermal performance for buildings has led to the almost universal use of blockwork for the inner leaf of cavity walls, replacing the common clay brickwork used previously. More recently, thin-joint aerated blockwork has been introduced. As well as high levels of thermal insulation, this form of masonry offers increased resistance to lateral wind loadings, reducing the need for wind posts in cladding panels, for example.

Modern thin-skin forms of cavity wall construction, with their relatively low embodied energies and high thermal efficiencies, are more environmentally friendly than the thicker types of solid masonry walling used in the past. They are, however, more complicated to construct due to the large amount of metalwork fixings they normally contain, and tolerances are often critical (Sutherland, 1988). In addition, thin cavity walls are more prone to thermal and moisture expansion/contraction than traditional, thicker forms of masonry. In order to accommodate movements, soft vertical and horizontal joints are now installed at regular intervals in brick, block and stone masonry walling. These, together with the almost universal use of stretcher bond for facing brickwork, tend to reduce the aesthetic appeal of masonry as a solid facing material.

11.9 Sustainability

Prior to discussing the sustainability of masonry materials and forms of masonry construction, it is useful in the first instance to consider what is actually meant by sustainability and to explain the importance of the construction industry to sustainable development, that is, the process by which sustainability is achieved.

11.9.1 Introduction to sustainability

Sustainability is a relatively recent concept that has arisen out of concerns about anthropogenic (ie, man-made) changes to the climate and the profligate use of the Earth's limited resources by the developed nations in particular. In essence, sustainability is concerned with promoting the most efficient use of resources, the protection of the environment and ecosystems, and the development of a more equitable world society generally. The different aspects of sustainability are often illustrated by means of a simple Venn diagram (Fig. 11.6).

As sustainability encompasses a wide range of complex yet interrelated issues, it is difficult to define in precise terms. Consequently, no single universally agreed definition has to date been adopted; instead, numerous definitions have been proposed, all of which are essentially vague in nature (Global Development Research Center, 2007). Probably the most commonly used definition is that of the Brundtland Commission in their 1987 report 'Our Common Future' (Brundtland, 1987): 'Sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs'.

While the economic and environmental issues associated with sustainability can often be quantified in terms of the different sciences involved (albeit often in a very

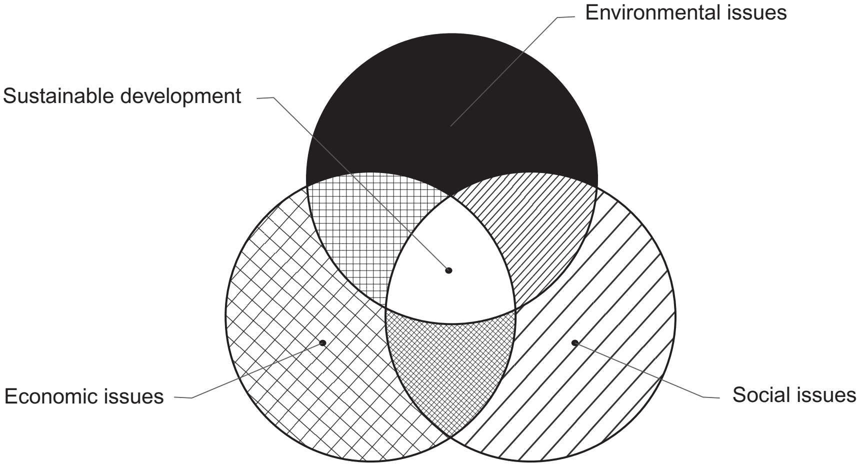


Fig. 11.6 Venn diagram to represent sustainability.

crude manner due to the complexities of the processes involved), the less tangible social aspects of sustainability, such as fairness and 'justice for all', are essentially qualitative in nature and cannot be quantified in the same manner.

Certain decisions relating to sustainability can, therefore, be made on the basis of quantitative scientific data. Others, however, may have to be based on *value-choice* judgments: for example, as a mark of a civilized society, we expect to live in a house or apartment even though cave dwelling may well be less damaging to the environment. The selection of sustainable products or processes therefore involves trade-offs between competing environmental, economic and social issues, and no perfect solution is usually possible. In addition, an accurate determination of what is truly sustainable, in an absolute sense, is usually not possible due to the current lack of understanding of the complex issues involved.

In practice, therefore, products or processes may simply be selected on the basis of environmental impacts (burdens) or economic costs (or some combination of these) over a specified lifespan—the product or process with the lowest overall impact, or cost, being considered the 'most sustainable' option. As such, sustainability reduces to a simple comparative exercise between alternatives, all of which may actually be unsustainable in absolute terms. In this respect, it should be noted that relative sustainability is not a valid concept, with terms such as 'more sustainable' and 'less sustainable' being considered essentially meaningless (Sutton, 2000). They are, nevertheless, still widely used.

On a more general note, it should be noted that most of the current tools for evaluating the environmental performance of construction materials and buildings are commercial in nature. This has led to the 'black box' situation, where it is often not possible for the client to verify the results obtained from these tools because data on the whole life performance of materials and their costs, for example, may not be available for inspection.

11.9.2 Sustainability and the construction industry

Sustainability is, as already noted, concerned with the more prudent use of natural resources and the protection of the environment. These are issues of direct relevance to the global construction industry—a consumer of large quantities of natural resources in the forms of energy, water, materials and land. In the UK alone, for example, the construction industry consumes over 420 million tonnes of raw material annually and generates over 90 million tonnes of waste, much of which is disposed of in landfill sites (Environment Agency, 2007). In addition, the energy consumed in building services, that is, lighting, heating and cooling of buildings, etc., accounts for approximately half of the UK's current emissions of carbon dioxide, while around 10% of UK energy consumption is used for the production and transport of construction products and materials (Thistlethwaite, 2004).

In order to promote more sustainable practices within the UK construction industry, legislation is gradually being introduced to improve the thermal efficiency of buildings, to reduce waste and to encourage the use of recycled and more environmentally friendly materials generally.

11.9.3 Masonry as a sustainable construction material

Buildings and civil engineering structures such as arch bridges built in stone or brick masonry are usually capable of lasting centuries with minimal maintenance. These are important criteria in relation to the environmental and economic aspects of sustainability (Your Home, 2005). In the UK, for example, there are approximately 40,000 masonry arch bridges in daily use on highways, railways and canals; most are over 100 years old, while some are over 500 years old (Mabon, 2002). In terms of their whole life maintenance, masonry arch bridges probably perform better than those built in reinforced concrete or steel. Work by Steele et al. (2002), in particular, has shown that maintenance of the brickwork in masonry arch bridges represents less than 0.5% of the environmental impacts of initial bridge construction over a typical 120-year design life.

In relation to buildings, around 25% of the 23 million residential properties in the UK are more than 160 years old (Department of Communities and Local Government, 2006a). Most of these are built out of brick or stone, or some combination of these materials. Despite their age, the vast majority of these older properties continue to satisfactorily perform the functions for which they were originally built. The English Housing Survey 2010–11 (Department of Communities and Local Government, 2012) reports that nearly 60% of all domestic properties are over 50 years old.

The highly sustainable nature of the traditional materials used in UK housing is recognized by the (Royal Town Planning Institute, 2004), who note that:

With the exception of the 'system' building of the 1960s, there is little evidence that the traditional materials used in house building in the UK—brick, stone, tiles, slates, etc.—are not sustainable. The large numbers of 19th century houses that are still in good structural condition show that, with a responsible level of maintenance, the traditional materials are capable of lasting well over 100 years. Where major demolition has taken place—such as inter-war, peripheral council estates—this has been primarily for social reasons rather than a lack of building integrity.

Masonry also performs well in relation to the less tangible aspects of a building's sustainability. These include its adaptability, 'loveability' (Sayce, 2002) and overall robustness. Masonry also has excellent resistance to fire and will not ignite, burn or emit toxic fumes when exposed to extreme heat.

The principal use for masonry in buildings is as external and internal walling. Typically, these account for only 5–15% of the total environmental impacts from materials over a 60-year period (Fig. 11.7). Floor finishes and upper floors, on the other hand, are responsible for 40–45%; that is, they are significantly more damaging to the environment than the masonry used in a building.

Houses built of masonry also retain value, with the UK housing stock being worth £4000 billion in 2006. This is equivalent to around 60% of the country's net worth, making housing, and by implication 'bricks and mortar', its most valuable asset (Office for National Statistics, 2006). In relation to the social aspects of sustainability, brick and stone masonry in particular are visually attractive materials, and buildings clad with an external skin of brick or stone contribute to the aesthetic sustainability of the urban fabric generally. In addition, an insurer may charge more for buildings insurance for a timber-framed property than for one built with masonry (Hall, 2007).

Overall, masonry cavity walling with a brick or stone external leaf and a thermally efficient aerated or aggregate blockwork inner leaf can be considered to be a highly sustainable form of building construction.

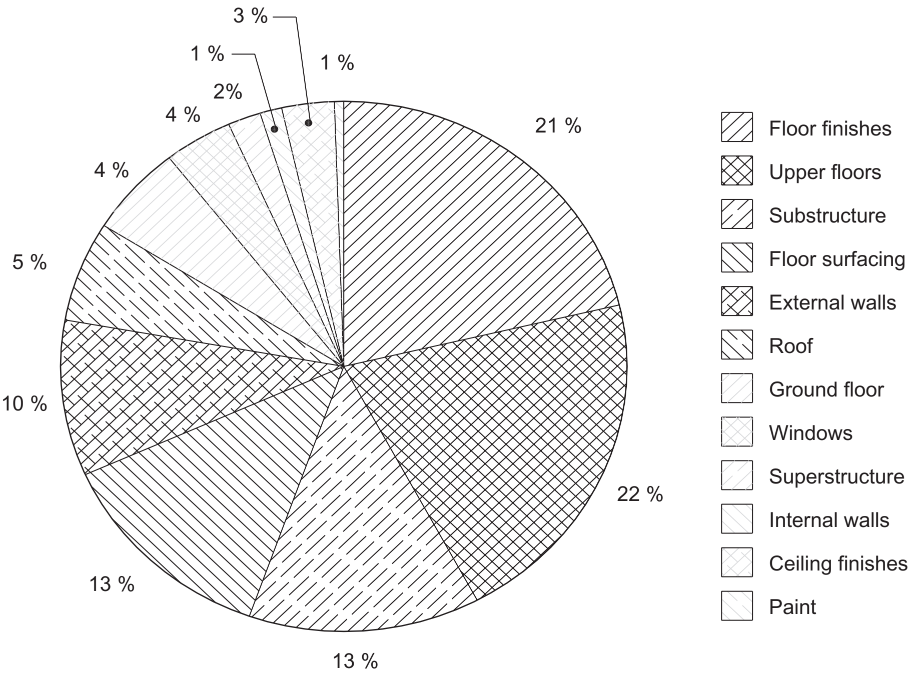


Fig. 11.7 Contribution of building elements to the whole life (60 years) environmental impacts of a typical building (Bown, 2007).

11.9.4 Quantifying the sustainability of masonry

In the UK most of the work on the sustainability of construction materials, including that of masonry and the different types of masonry wall construction, has been undertaken by the BRE. Their Environmental Profiles Methodology (Howard et al., 1999), originally developed in the late 1990s and subsequently updated, provides a means of comparing the environmental performance (impacts, or burdens) of over 250 materials and components used specifically in buildings. (NB, by themselves most construction materials do not have a specific function. It is only possible to quantify their sustainability when they form part of a component within a building, eg, an internal or external wall, or a roof.)

Environmental impacts were quantified in relation to a predefined set of 13 issues, including:

- climate change;
- ozone depletion;
- air pollution;
- fossil fuel depletion and extraction;
- mineral and water extraction;
- waste disposal;
- transport pollution and congestion: freight.

Three types of profiles were developed.

1. 'Cradle to factory gate' performed for a standard mass of a product or material (eg, 1 tonne of bricks), without any concern for their eventual use.
2. 'Cradle to installed-on-site' for walls, floors, ceilings, etc., performed on a per-square-metre basis.
3. 'Cradle to grave' for building elements with a 60-year lifetime only, taking into account maintenance, replacements and disposal, again on a per-square-metre basis.

The profiles were then used in a range of BRE publications and tools for assessing the environmental and economic performance of different types of buildings. These include:

- the Green Guide(s) to Specification (different versions are available);
- ENVEST software package;
- the BREEAM suite of assessment methods and tools and
- the Code for Sustainable Homes.

11.9.4.1 Green Guide(s) to Specification

Here the principal elements within buildings—floors, walls, roofs—are assessed in terms of a simple A–B–C rating system, with an 'A' having the least environmental impact. (NB: The revised 2007 version of the Green Guide, yet to be published, adopts an A*–E rating system.) Table 11.6 shows the ratings for a variety of external masonry walls. It can be seen that the brickwork and blockwork cavity walls listed achieve an 'A' summary rating, whereas the cavity wall with a reconstituted stone outer leaf receives a 'B' rating. It should be noted that the Green Guides are a simplistic tool for enabling comparative assessments between preselected elements of construction only, as an 'A' rating for one form of construction or element may not be equivalent to an 'A' rating in another element for the same environmental impact.

Table 11.6 Extract from a rating table in the *Green Guide to Specification* (Anderson et al., 2002)

Traditional forms of cavity wall construction	**Summary rating**	**Climate change**	**Fossil fuel depletion**	**Ozone depletion**	**Human toxicity to air and water**	**Waste disposal**	**Water extraction**	**Acid deposition**	**Ecotoxicity**	**Eutrophication**	**Summer smog**	**Minerals extraction**	**Cost (£/m^2)**	**Typical replacement interval**	**Recycled input**	**Recyclability**	**Recycled currently**	**Energy saved by recycling**
Element																		
Brickwork outer leaf, insulation, aerated blockwork inner leaf, plasterboard/plaster	A	A	A	A	A	B	A	A	A	A	A	A	55–105	60	C	A	B	A
Brickwork outer leaf, insulation, dense blockwork inner leaf, plasterboard/plaster	A	A	A	A	A	C	A	A	A	A	A	A	55–105	60	C	A	B	A
Fair-faced reconstituted stone outer leaf, insulation, dense blockwork inner leaf, plasterboard/plaster	B	A	A	A	A	C	A	A	C	B	A	B	125–150	60	C	A	B	A

11.9.4.2 ENVEST software package

A more comprehensive assessment of the whole life environmental and economic performance of buildings is provided by the BRE's ENVEST software package (Building Research Establishment, 2007a). This gives an indication of a building's operational energy and whole life costs, and allows designers to investigate the trade-off between the environmental impacts of the different elements in a building and operational energy.

ENVEST expresses environmental performance in terms of 'ecopoints' (notional units, in which 100 ecopoints represent the environmental impact of one UK citizen over a period of 1 year). The validity of this simplified approach to environmental performance, that is, reducing the 13 impact categories to a single point 'score' is, however, questionable as it involves the subjective process of weighting (International Organization for Standardization, 2006).

Fig. 11.8 shows the results of a recent ENVEST analysis to quantify the total 85% energy consumption of a typical four-bedroom residential property in the UK over a 100-year lifespan. The external walls were of brick/block cavity construction and all main elements—walls, roof, etc.—complied with the 2005 UK Building Regulations governing thermal performance. It can be seen that the embodied energies of all the materials used in the building are low in relation to the operational energy, with external masonry walls accounting for less than 2% of total energy.

11.9.4.3 BREEAM and the Code for Sustainable Homes

In addition to the environmental impacts of the materials used in the fabric of buildings, the BREEAM suite of assessment methods (Building Research Establishment, 2007b) and the recently introduced Code for Sustainable Homes (Department of Communities and Local Government, 2007) assess building performance across a range of other categories. These include management, operational energy, health and well-being, pollution, land and water use and ecology. Credits are awarded according

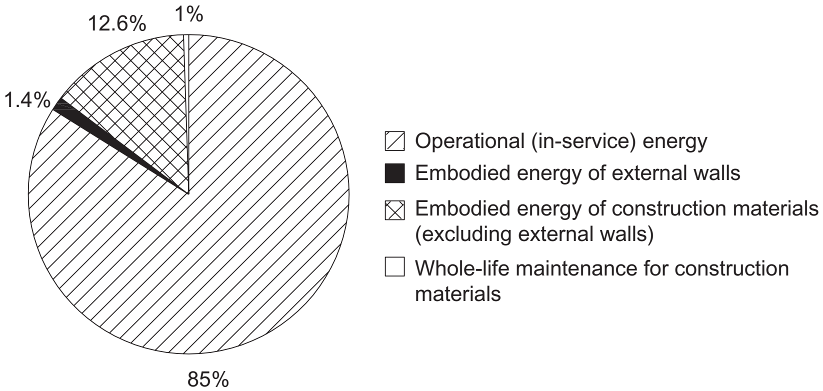


Fig. 11.8 Comparison of embodied energy of the construction materials and operational (in-service) energy for a four-bedroom residential property at 100 years, according to ENVEST (Bown, 2007).

Table 11.7 **Environmental weighting factors in the Code for Sustainable Homes (Department of Communities and Local Government, 2007)**

Environmental impact categories	No. of credits in each category	Environmental weighting factor (as % of total possible points score available)	Points score for each credit in category
Category 1–Energy and CO, emissions	29	36.4	1.26
Category 2—Water	6	9.0	1.50
Category 3—Materials	24	7.2	0.30
Category 4—Surface water run-off	4	2.2	0.55
Category 5—Waste	7	6.4	0.91
Category 6—Pollution	4	2.8	0.70
Category 7—Health and well-being	12	14.0	1.17
Category 8—Management	9	10.0	1.11
Category 9—Ecology	9	12.0	1.33
Total	104	100.0	–

to performance, with minimum standards specified in each category. Using a set of environmental weighting factors, these are converted to produce a single overall score. In the case of BREEAM, buildings are rated on a scale of Pass, Good, Very Good or Excellent, and a certificate awarded. The Code for Sustainable Homes, on the other hand, has six levels of award, Level 6 being an aspirational standard based on zero carbon emissions for a dwelling.

Table 11.7 shows the environmental impact categories and associated weighting factors considered in the Code for Sustainable Homes. It can be seen that operational energy/CO has the largest weighting (36.4%), with that for materials being relatively low (7.2%). Although these weighting factors are essentially subjective in nature, they do nevertheless illustrate that reducing the operational energy of a building over its lifespan is considered to be significantly more important, in terms of sustainability, than the environmental impacts of the materials used in the fabric of the building itself.

11.9.4.4 Environmental product declaration

In 2012, then again with amendments in 2013, a new European Standard, BS EN 15804:2012 + A1:2013 Sustainability of construction works, Environmental product declarations (EPDs)—Core rules for the product category of construction products, was published. This standard was developed in order to provide guidance to the construction product standards committees within CEN, on how to develop a harmonized approach to the development of product category rules (PCRs) that would enable EPDs to be generated by manufacturers.

The standard draws together the principles of LCA based on the underpinning principles of ISO 14040, ISO 14044 and ISO 14025, and applies an assessment of the 24 impact categories and applies these to construction products, construction services and construction processes in such a way that the results can be verified.

These principles also link into the development of standards that can be used to make environmental declarations for 'construction works', that is, the buildings, through a suite of standards including EN 15643-1 and EN 15643-2 that relate to the assessment of buildings and their performance.

There are three options with regards to reporting the environmental performance of the construction product which reflect different cutoff points in the life cycle. The minimum mandatory requirement in the standard required a declaration for 'cradle to gate' that includes the impacts of the raw material extraction, transport of the raw materials to the manufacturing site and then the manufacturing process itself. These stages fall within the 'Production Stage' A1–A3 in the lifecycle stages. The second option for declaring a product's environmental performance is known as 'cradle to gate with options' which allows for the inclusion of selected 'stages' beyond the factory gate, and typically includes the transportation of the construction product to the customer, Stage A4 in the Construction Process stage. The all-inclusive, full life-cycle option is the 'cradle to grave' option, that brings in the need to develop scenario-based impacts that relate to the construction process stage A5, the Use Stage of the building, stages B1–B7 and the End of Life Stage, stages C1–C4, and the option to declare 'Beyond end of building life' in Stage D.

Table 11.8 shows the 24 'impact' parameters to be calculated and reported within the context of the standard, and the stages, cradle to grave, etc., are to be included in the LCA calculations. The main seven impact parameters fall within the Environmental Impacts, which includes commonly reported parameters such as Global Warming and Ozone Depletion.

As only EPDs created and reported using the same PCR can be used for like-for-like comparisons, the production of EPDs at product level is not designed to be used for direct comparisons; however, the use of the impact parameters in building information modelling (BIM) design packages does allow designers and architects, at a design stage, to compare building level environmental credentials, based on construction product choices and combinations.

11.9.4.5 Circular economy

Within the concept of resource efficiency (RE) and general sustainability principles, the growth of the interest around the *circular economy*, particularly in the context of consumer goods, continues to grow. Organizations such as the Ellen MacArthur Foundation (http://www.ellenmacarthurfoundation.org) have developed new business models based on the principles of a circular supply and recovery of materials, rather than the traditional linear supply chain (cradle to grave), or the take-make-dispose, the aim being to maximize the amounts of materials that are retained within the make and use stages of the life cycle of a product (cradle to cradle), rather than at the end of life going down the disposal route.

Table 11.8 **Environmental impact parameters reportable within EN 15804 (year)**

Environmental impacts	**Resource use**	**Waste categories**	**Output flows**
Global warming	Renewable primary energy	Hazardous waste disposed	Components for reuse
Ozone depletion	Renewable primary energy as raw material	Nonhazardous waste disposed	Materials for recycling
Acidification of soil and water	Total renewable primary energy	Radioactive waste disposed	Materials for energy recovery
Eutrophication	Nonrenewable primary energy		Exported energy
Photochemical ozone creation	Nonrenewable primary energy as raw material		
Depletion of abiotic resource—elements	Total nonrenewable primary energy		
Depletion of abiotic resource—fossil fuels	Secondary materials		
	Renewable secondary fuels		
	Nonrenewable secondary fuels		
	Net fresh water		

Circular economy models work very well for a wide range of clothing, packaging and consumer goods, especially electrical and white goods. This is as a result of the relative high intrinsic material value of either components or raw materials, precious metals, rare earth metals, etc., as seen in mobile phones, computers, and for products that have a relatively short use phase of the life cycles, typically 3–5 years or less. However, for construction materials, and specifically masonry, the business model for a circular economy has yet to be refined to be able to accommodate the relatively low 'intrinsic value' of the construction products (bricks, blocks, mortar) themselves, as well as the long 'in use' phase of the life cycle. Typically masonry will spend 60–150 years in service; therefore there is little or no incentive for a 'manufacturer' of these types of products to recover units and 'process' them ready for resale and reuse.

It is not true to say that no recovery of masonry units and reuse occurs; there are a wide range of merchants and reclamation specialists that trade in 'salvage' and recovered demolition materials. Current business models for this 'salvaged' material typically require a significant cost associated with manual sorting, cleaning off mortar and packing, far outweighing the actual value of the material. Only where the selling price driven by uniqueness or scarcity drives the market and covers these labour costs is there an incentive to recover for reuse.

On a technical level, reuse of bricks and natural stone units typically comes with no technical specification or guarantees. Often bricks recovered and reused that have been in service in protected (sheltered or internal) walls that subsequently get used as external facing bricks, can result in problems, frost damage and efflorescence being the main ones typically reported. These types of reused masonry units fall firmly within the concept of *caveat emptor* (buyer beware).

The main markets for masonry materials in this area are certain brick types, Victorian and London Stocks, and natural stone masonry units. In the main, most masonry (bricks, blocks, mortar) ends up as recycled aggregate, where the masonry is crushed and screened, either on site or off site, and supplied back as piling mats or general bulk fill aggregate.

11.9.5 Masonry and the design life of buildings

The BRE Green Guides assume a design life of only 60 years when evaluating the whole life environmental performance of buildings. After this, the buildings are assumed to be demolished and the various materials are disposed of as landfill, or recycled/reused to some degree. This approach is overly simplistic, however, as it fails to recognize that buildings may have different lifespans depending on their use and, indeed, reuse. It has been suggested, for example, that new housing should be designed to last a minimum of 200 years (Green Guide Update, 2006). Masonry would appear to be an ideal material in this respect as it is capable of lasting centuries. Finally, it is of interest to note that some of the lightweight modular designs in the recent UK Design for Manufacture £60,000 Home Competition (Department of Communities and Local Government, 2006b) may have lifespans of only 30 years (Building Products Magazine, 2007).

11.9.6 Whole life costs of masonry

Whole life costing considers the cost of a building over its entire life and includes initial build costs, maintenance and eventual demolition. In practice, durable, long-life buildings that require little maintenance may be better value and more sustainable overall than designs that are cheaper to construct but have maintenance costs. Structures built in masonry perform well in this respect as they require minimal maintenance. Typically, this only involves repointing or replacement of brickwork or stonework (Fig. 11.9). Examples of the whole life costs of two types of masonry walling for different lifespans are shown in Table 11.9.

11.9.7 Reclamation and recycling of masonry

Reclamation and recycling of construction materials from demolition sites is an important aspect of sustainability. Data on the amount of masonry that is reclaimed or recycled in the UK are, however, limited. A 1998 survey estimated that 147 million reclaimed bricks were sold each year, with a value of £80–160 million (Salvonews, 2006). The quality of reclaimed bricks was, however, poor with only 5% having

Fig. 11.9 Example of replacement brickwork.

Table 11.9 Whole life economic performance of masonry walling (2005 base prices) (Bown, 2007)

	Initial cost (£/m²)	60 years (£/m²)	100 years (£/m²)	150 years (£/m²)	300 years (£/m²)	500 years (£/m²)
225 mm solid brickwork	81.41	162.17	191.96	260.93	455.54	697.28
102.5 mm brick/100 mm	96.73	179.09	221.78	324.78	–	1124.93

Block cavity wall.

any frost resistance guarantee. The survey also found that between 600 and 1200 million bricks were crushed and used annually as low-quality hardcore fill or capping layers in road construction. Table 11.6, from the *Green Guide to Specification*, gives a broad indication of the recyclability and energy saved by recycling masonry walling while the issues associated with the use of reclaimed bricks are discussed in more detail by the UK Brick Development Association Ltd (Brick Development Association, 2001b).

A thriving market exists for reclaimed stone products in areas of the UK where these have traditionally been used for building work, for example, the Cotswolds and parts of Yorkshire. Much of the reclaimed stone is used for small extensions to existing residential properties, with its cost invariably being higher than that of newly quarried stone.

In practice, while soft lime mortars may readily be removed from bricks and stone, cement-based mortars cannot be so easily, or effectively, cleaned off. The reclamation of bricks that have been laid in cement-based mortar is consequently more problematic and may not be feasible in terms of its cost-effectiveness or environmental impacts at the present time.

Brick, block and stone masonry from demolition sites may be processed and reused (usually in combination with crushed concrete) as a coarse aggregate for making new concrete. Its use in this way within the UK has, however, been very limited to date due to concerns about the variable composition of recycled demolition and excavation wastes and the possible presence within these wastes of undesirable substances such as plastics, wood, ferrous metals and gypsum. Careful separation of the different materials is therefore essential at recycling centres, together with adequate washing to remove soil, silt and clays (WRAP, 2007). Ideally, wastes should be sorted into colour-coded stockpiles:

- black—asphalt;
- white—concrete;
- red—mixed waste, but generally composed of brick.

In general, the strength of concrete containing recycled aggregates (RA) is lower than that of concrete made with natural aggregates. When used at moderate levels, that is, up to 20% replacement by mass of natural coarse aggregate, the effects of RA on the overall performance of concrete, and the proportion of brick within the RA, are not considered significant (WRAP, 2007). When the coarse aggregate consists entirely of RA, the strength of concrete decreases with increasing brick content, with concretes containing 100% crushed brick having strengths approximately 20–25% lower than those made entirely with recycled concrete aggregate (RCA). The general effects of RA, and the brick content of RA, on the strength of concrete are shown in Fig. 11.10 for different classes of concrete exposure. In practice, to achieve the same cube strength and consistence as concrete made with natural aggregates, it may be necessary to increase the cement content of RA concretes or to use large dosages of admixtures, both of which are undesirable in terms of their environmental impacts (Hansen, 1992; WRAP, 2007).

Crushed brick can also be used as a fine aggregate in concrete. In practice, however, this can significantly increase water demand, leading to low-strength concrete (WRAP, 2003). The effect on the performance of concrete is similar to when brick is used as a coarse aggregate; that is, the 28-day strength of concrete decreases with increasing brick content (Khatib, 2005). At later ages, however, the rates of gain of strength can be higher in concretes containing crushed brick fine aggregate, due possibly to pozzolanic action between the crushed brick and the products of cement hydration (Wild et al., 1996).

When ground to a very fine powder, clay bricks exhibit pozzolanic properties. Waste bricks, in particular, have the potential to be used as a partial replacement for cement in mortars and concretes, in a similar manner to fly ash (PFA) or silica fume. The benefits of this include reduced permeability and greater resistance to sulphate attack and alkali-aggregate reactivity (Gupta, 1992). Replacement of 10% cement by ground brick was found to have no effect on the compressive strength of mortar while tests on concrete showed that up to 20% replacement of cement was possible with no detrimental long-term effects (Golaszewski et al., 1999).

Further details of the performance of concretes containing recycled masonry materials, and proposed classes of RA, are available from (WRAP, 2007).

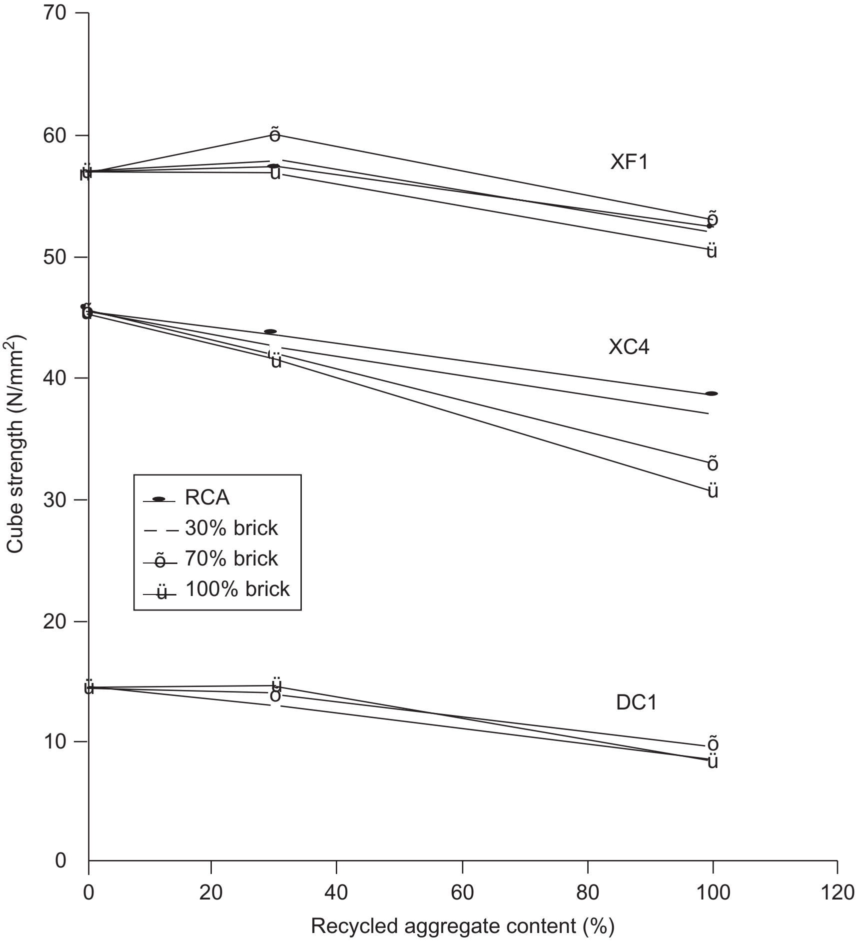


Fig. 11.10 Effect of RA, and the brick content of RA, on the strength of concrete for different classes of exposure (courtesy of WRAP, 2007) (XF1, freeze/thaw attack—moderate water saturation without de-icing agent; XC4, carbonation—moderate humidity or cyclic wet and dry; DC1, design chemical class 1).

11.9.8 *Thermal mass*

Clay bricks, dense aggregate blocks and stone units have high thermal mass. Buildings containing these types of masonry take a long time to heat up and a long time to cool down. As a result they have a relatively steady internal temperature, which reduces the amount of external heating or cooling they require. Recent research has, in fact, shown that medium-weight and heavyweight masonry and concrete homes should have lower total energy consumption and CO_2 emissions over an assumed 100-year life than lightweight timber homes (Concrete Centre, 2006). This is due to the thermal mass of the blockwork walls and concrete floors, which restricts overheating during the summer months and captures solar gains on winter days.

11.10 Examples of sustainable masonry construction

11.10.1 BedZED, south London

The Beddington Zero (Fossil) Energy Development (BedZED) is a mixed-use scheme in south London consisting of 82 homes and 3000 m^2 of commercial or live/work space that was completed in 2002. It was initiated by the BioRegional Development Group and Bill Dunster Architects and was developed by London's largest housing association, the Peabody Trust. BedZED was designed as a carbon neutral development, that is, one that produces at least as much energy from renewable sources as it consumes—with no net addition of carbon dioxide to the atmosphere. A particular aspect of the development is that it employs proven materials, including brick and block, and relatively low-tech methods of construction (Fig. 11.11).

In order to maximize solar gain, the terraced properties face south and have double- or triple-glazed windows with low-emissivity glass. The walls are a mixture of brick and dense aggregate block cavity construction and timber stud weatherboarding with cavity blockwork, with a 300-mm-wide cavity filled with insulation. The large thermal mass of the dense blockwork inner leaf and precast concrete floors allows heat to be stored during warm periods and warmth to be radiated during cooler periods, significantly reducing the amount of external heating or cooling required.

Although the BedZED scheme has suffered a number of operational problems since its completion in 2002, these relate principally to the biomass-fuelled system for producing heat and electricity (combined heat and power—CHP) and the reed beds for filtering sewage water for use in toilets and gardens (Slavin, 2006). The buildings themselves, on the other hand, appear to be functioning well and while the development is not yet carbon neutral, heating requirements for BedZED homes are only around 10% of those of a typical home (Ecozine, 2007).

Fig. 11.11 BedZED.
Courtesy of the BioRegional Development Group.

As the movement of construction materials around the UK accounts for about 30% of all road freight, a policy of local sourcing of materials was adopted at BedZED. This resulted in some 120 tonnes of carbon dioxide emissions being saved, equivalent to 2% of the embodied carbon dioxide of the BedZED buildings (Green Building, 2007).

11.10.2 Winterton House, London

Winterton House is a 23-storey residential tower block in the Tower Hamlets area of London. Built in 1968, it consisted of a lightweight steel frame with a central reinforced concrete service core. Lightweight precast concrete units were used for the floors with glass-reinforced plastic (GRP) panels for cladding. The original building quickly suffered from a variety of problems. These included ingress of water through the mastic joints, causing corrosion of the steel reinforcement in the floors, together with excessive sound transmission between rooms and the presence of asbestos as a means of fire protection. Three similar buildings erected around the same time also suffered with these problems and were demolished.

Instead of demolishing Winterton House, it was decided to undertake a major refurbishment of the property. In the mid-1990s the building was therefore stripped back to the bare steel frame and reinforced concrete floors installed. It was then reclad with a freestanding outer skin of load-bearing brickwork (Fig. 11.12). Finally, the extra weight imposed on the frame by the new concrete floors was 'pulled out' of the steel columns and transferred across to the perimeter brickwork via a steel transfer framework at

Fig. 11.12 Winterton House (© Tom Chance/Bioregional).

roof level. This involved the use of hydraulic jacks to prestress or ‘prestrain’ the brickwork, which then acted compositely with the steel frame of the building.

In addition to acting as a weatherproof cladding, the clay brickwork at Winterton House contributes to the load-bearing capacity of the building. This is radically different from the manner in which clay brickwork cladding is normally used on tall buildings, that is, as a non-load-bearing rain screen, and can lead to thinner structural frames being required. Winterton House also demonstrated that masonry can be successfully employed to prolong the life of existing buildings, a key aim of sustainable construction generally. Other benefits of this cost-effective form of off-the-frame brickwork cladding include greater building robustness and, as the brickwork is generally thicker, the use of alternative bonding patterns and arch features to improve the visual appeal of buildings (Bingel, 2001).

11.10.3 Queen Square, Leeds

Queen Square is one of only two Georgian squares in Leeds still in their original state. Most of the square was built between 1803 and 1815 and was planned as three terraces round a central garden. The grade two listed properties were constructed of red brick with stone details, and consist of two- and three-storey buildings with basements and warehouse facilities (Fig. 11.13). Originally occupied by local, wealthy wool merchants and solicitors, they have been used for a variety of purposes during their 200-year lives. These include schools, shops, private apartments and offices for various businesses and local societies. The properties are now part of Leeds Metropolitan University which has recently invested more than £1,000,000 in their refurbishment to provide new office accommodation and teaching space. Queen Square is an excellent example of how masonry buildings can be successfully reused, thereby avoiding the need for demolition and at the same time contributing to the attractiveness of the urban environment.

Fig. 11.13 Queen Square, Leeds.

Fig. 11.14 Swaffham Community Centre, Norfolk.
Courtesy of H+H Celcon.

11.10.4 Community Centre at Swaffham, Norfolk

This £1.5 million, low-energy building consists of two large halls with adjoining kitchen and café together with offices and is used for training, meetings, social events and as a crèche (childcare center). Thin-jointed Celcon Solar Plus Aircrete blocks form the entire envelope of the building and achieve the required U-value without the need for a cavity, insulation, ties or other components, that is, simple construction. The blocks were easy to lay and there was little wastage. The cladding is a combination of cedar boarding and Sto render. The building also features high-performance windows and an air-handling system for fresh air and ventilation with a heat recovery system. An unusual feature is the sedum roof, a 'living' or 'green' roof, the vegetation providing insulation and protection from ultraviolet light (Fig. 11.14).

11.11 Future trends

As part of the drive towards more energy efficient buildings, the UK masonry industry has introduced aerated blocks and cavity walls with wider cavities and increased amounts of insulation. The development of new, more thermally efficient products and forms of construction will continue as the regulations governing the operational energy of buildings are progressively tightened in future years. In addition, types of masonry products and forms of construction already in use in mainland Europe (Fig. 11.15) are likely to find their way onto the UK market. These include thick single-leaf walls built from highly perforated clay block units and subsequently sprayed with external insulating render. Greater use of unfired-clay bricks (using innovative methods such as shelled blocks where the outer part is made with cement added while the inner core is made of earth. I could provide references or refer to Chapter 21) is also likely, as approximately 85% of a fired brick's embodied energy is due to the firing process itself.

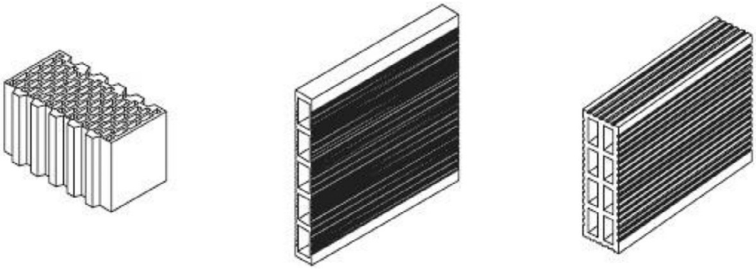


Fig. 11.15 Highly perforated clay block units.

Offsite manufacturing is also an area in which the masonry industry has, to date, made relatively little progress. This is due, in part, to the brittle nature of most forms of masonry construction and the cost-effectiveness of masonry prefabrication generally. The use of aerated blocks and thin-joint mortar systems for prefabricating lightweight walls that can then be transported to site appears to have significant potential in this respect.

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Sustainability of natural stone as a construction material

12

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12.1 Introduction

This chapter presents an overview of the use of natural stone as a building material, giving specific regard to its place in sustainable development. The major building stones of sandstone, limestone, granite and slate are examined. Sustainability has now become an integral consideration and is a permanent aim for the construction, operation, maintenance and decommissioning of buildings. Stone masonry is technically durable, structurally robust, beautiful and capable of offering a breadth of properties which make it the ideal building medium as part of sustainable development, not least its security, fire safety, acoustic, thermal and moisture-regulating characteristics.

A brief overview of past, present and future build contexts is presented. Focus is placed on the material characteristics, in particular the unparalleled durability of natural stone; this is coupled with an introduction to issues of decay. Practical measures are suggested to support the reader in specifying natural stone with confidence. The repairability of stone masonry is also presented as a measure of durability, and consequently an element of sustainability, in its own right.

The performance of natural stone as a sustainable building material is appraised in terms of the embodied energy (EE) and specifically embodied carbon (EC) impact of the initial build. This impact is appraised alongside the operational efficiency expected of the structure, and the whole-life impact of building with stone is also discussed. The sustainability of natural stone in building reaches further than strictly environmental terms: less measurable, community-amenity/sociological benefits are also intimated.

Masonry has been astutely demonstrated to represent a key choice in sustainable building (Key, 2009): If this is true of brickwork, surely the case for natural stone masonry is even stronger. The chapter concludes, 'Why not use stone?' The primary factors militating against its use have been initial cost, lead times, and specifier confidence. However these factors are dated, and changing. Natural stone in building is highly sustainable on the grounds of incredibly low environmental impact, renowned community-amenity/sociological benefits, and is financially competitive when measured on a whole-life basis.

Sustainability of Construction Materials. http://dx.doi.org/10.1016/B978-0-08-100370-1.00012-3

12.2 Stone exploited

12.2.1 Categories of use

Natural stone is a versatile medium, suitable for working to a variety of applications. The typical applications holding market share across the UK stone industry include:

- Aggregates
- Cladding/façade masonry
- Conservation of existing structures
- Flooring
- Garden landscaping
- Internal fixtures
- Kerbing and paving
- Roofing
- Structural masonry

This chapter deals primarily with dimension stone for building, principal applications being cladding and structural masonry. Slate for roofing is also discussed. Typical building stone applications are indicated in Fig. 12.1.

12.2.2 Historical use of stone

Natural stone is a widely used building material throughout the British Isles, and globally exploited as such. Knowledge of stone in use was once widespread, with both crofter and master mason alike exploiting it in building. Empirically developed over centuries of use, stone's material characteristics were well appreciated: high strength in compression yet incapable of supporting appreciable tension/flexural loadings, stonework was exploited accordingly. The masonry arch demonstrates this well: By its geometry, the arch works wholly in compression, designing out the tension limitations, and playing to the strength of the material. Fig. 12.2 represents the maxima and

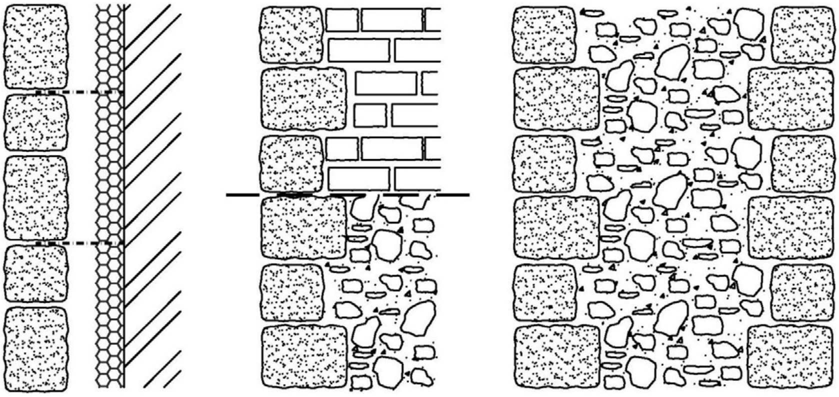


Fig. 12.1 Typical construction configurations of stone masonry in modern and traditional structural contexts.

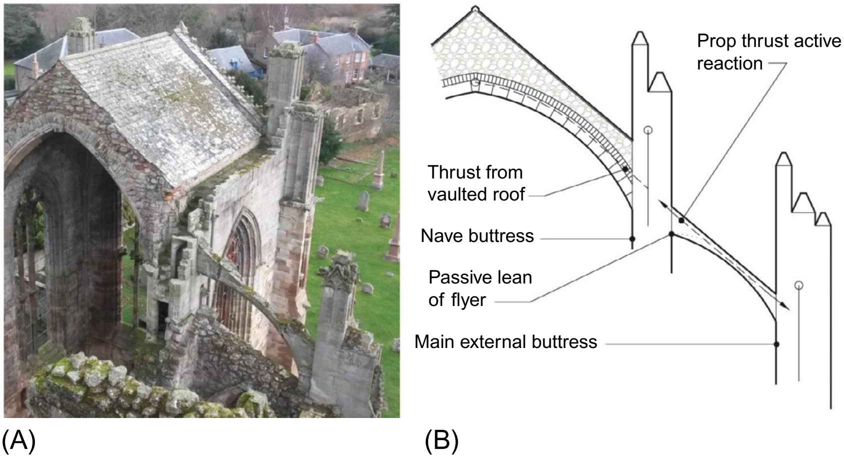


Fig. 12.2 The masonry arch in practice: (A) vaulted stone roof and flying buttress; (B) schematic structural functions.
From Klemm, A.J., Wiggins, D.E., 2015. Lime mortar and sacrificial protection of heritage stonework. Proc. Inst. Civil Eng. Eng. Hist. Herit. 168 (EH4), 167–176.

minima of possible lines of thrust within an arch, infinite range in between; a qualifying assumption in historical masonry was that it has no tensile strength (Heyman, 1995). Quality in historic stone masonry buildings used to be a major determinant: Heritage structures were on the whole built with pride, to last. A lengthy service life was held as a measure of quality (Hutton and Roston, 1997).

Prior to cheap fuel now permitting manufacture of materials demanding high energy costs in production, natural stone was the building material of choice. The advent of cheap fuel and transport drove the vernacular (local distinctiveness) almost to extinction. Small-scale local quarries and kilns could not compete with centralised industry. Sense of place is only now reemerging in significance for development, in the aftermath of the stigmatised system-builds of the last century. The stone industry in the United Kingdom has been kept alive largely due to the long-term demand from the heritage sector, and from sporadic, one-off, high-value projects. The decline in the use of natural stone in building has nothing to do with the outworking of resources.

12.2.3 UK stone resources

Globally, stone is probably being formed in the Earth's crust faster than it is being exploited (Stone Federation Great Britain, 2011). The stone resources of the United Kingdom are presented at a broad scale in the British Geological Society's (BGS's) map (British Geological Society, 2001), although many more small-scale quarries are to be found across the United Kingdom (McMillan et al., 2006). The variety of building stones within the United Kingdom is remarkable (eg, McMillan, 1997). An issue with this variety can be its accessibility, and many historic quarries lie dormant, not due to overexploitation but due to the collapse of the stone industry early in the 20th century. The reopening of some historic quarries remains the subject of contemporary discussion (Yates, 2005).

Quantifying stone reserves is difficult, not least because of the commercial sensitivity tied up in it. Rock deposit geometry can vary substantially between stone types, and volume estimation is therefore challenging. Stone reserves within the United Kingdom are nevertheless considered to remain 'significant' (British Geological Society, 2005). In short, the general view across the UK stone industry is that it is the demand that is small; supply is abundant.

12.2.4 Extraction and processing

Historically, quarrying was very much a local task. This fed the development of the vernacular, local distinctiveness, certainly before transportation became widespread and economical. Local sourcing of stone markedly influences its sustainability credentials, with transportation within the United Kingdom accounting for around 10–20% of the EC (comparing Cradle-to-gate (C-G) and Cradle-to-site (C-S)). Importation increases the carbon footprint many times over (Crishna et al., 2010). Local sourcing supports employment, often rural. Energy sources associated with extraction and processing include fuel for plant, modest use of explosives, and electricity and water for processing.

The extraction and processing of dimension stone is fairly consistent in terms of process across the United Kingdom. Extraction processes vary according to the type and characteristics of the stone; however, in the main, the aim is to secure the largest bulk block size within practical constraints. These blocks are then inspected to appraise the most efficient way of cutting into slab form with minimum wastage (Stark, 2005). Typically the stone is seasoned in the yard to harden up, although it may be processed 'green'. Cutting is by plant machinery, the primary cut being to reduce the rough bulk to slab forms, and the secondary cut(s) to dimension stone sizes. Tooling, dressing and other finishing is then undertaken according to the final product required.

Approximately one-third of the rock deposit is estimated to become the primary product of dimension stone, the rest of which comprises overburden or primary waste, which then becomes available for by-product usage (Siegesmund and Török, 2011). This general approximation is of course dependent on the type of stone being quarried, and the product required.

12.3 Material characteristics

12.3.1 Overview

The three major stone categories are sedimentary, igneous and metamorphic, listed in Table 12.1.

The sedimentary stones have been chiefly used in building in the United Kingdom as dimension stone, owing largely to their ease of extraction and working, relative to the other categories. Historically, igneous and metamorphic stone in building tended to comprise field boulders or random rubble. It was only after the advent of steam power that granite, for example, came into prominence as a dimension stone for building. Historical examples can be found which combine the different categories.

Table 12.1 **Building stone categories**

Category	Building stones
Sedimentary	Sandstone, limestone
Igneous	Granite, basalt
Metamorphic	Slate, marble

The dense, squared rubble of the main structure at Auchindoun Castle, Grampian (Scotland, UK; c.1480) is ornately picked out with special units, which required extensive working, carved from sandstone (Fig. 12.3A, note the tas-de-charge at the vault springer). Another example is demonstrated at Smailholm Tower, Borders (Scotland, UK), where the main structural material is whinstone, and the quoins, lintels and ingoes are built of sandstone permitting more formal dressing (Fig. 12.3B).

12.3.2 Sedimentary

Sandstones are a 'clastic' type of rock; that is, they are formed from fragments of rock from earlier rock formations which have weathered away. They comprise mineral grains of sand/sediment, deposited and compacted under water, which then become cemented to form rock. Quartz is the major constituent, the typical grain material, hard and chemically stable. Owing to the deposition process, the resulting rock microstructure is anisotropic, and typically has different characteristics per plane considered. The degree of compaction during initial formation dictates in many respects the density of the grain packing, and in so doing notably influences the final porosity and microstructure.

Fig. 12.3 (A) Auchindoun Castle (Scotland, UK): squared rubble walling material with dressed sandstone elements. (B) Smailholm Tower (Scotland, UK): whin walling picked out with sandstone decoration.

The grain size is typically some 0.06–2.0 mm (Ashurst and Dimes, 1984). The grains do not melt or amalgamate as in igneous/metamorphic rocks, and they are packed according to the deposition process; the cementation takes place either through disruption and redeposition of the quartz around the grain edges during compaction, or through cementing minerals suspended/dissolved in the percolating water (Blyth and De Freitas, 1984). Durability of sandstones is determined not by the hardness or chemical stability of the major constituent (quartz grains), but by the cementing agent which binds them. The cementing mineral is used as a sliding-scale durability measure, with siliceous as most durable, followed by calcareous (calcite being the mineral binder), ferruginous (iron oxide) and at least durable argillaceous (clay binder). It is usual to find sandstones with binders representing a blend of minerals, the classification being by the predominant binding mineral (Leary, 1986).

The major constituent in limestone is calcium carbonate. An amount of siliceous material is usually present, generally in the form of quartz grains interspersed. A degree of clayey impurities can also be found, which dictates the hydraulicity of the limestone if burned for quicklime. Limestone is similar to sandstone in that its microstructure is comprised of grains of matter, cemented by a mineral binder. However, in a limestone, the predominant proportion of grains are comprised of fragments of marine skeletons, shells and fossils (Blyth and De Freitas, 1984). Both the grains and the cementing agent comprise calcium carbonate.

The microstructure is generally developed through calcitic precipitation/crystalline growth surrounding the grains; the cementing calcite fills the interparticle voids originally present on deposition. Curiously, however, this does not of itself lead to a lesser porosity than a sandstone: indeed, the sandstones on the whole have a lower porosity than limestone (Table 12.2). This is because the cementing agent calcite is itself highly

Table 12.2 **Physical characteristics of selected building stones**

Characteristic	Sandstone	Limestone	Granite	Slate
Density (bulk specific gravity) (Mg/m^3)	2.00–2.35	1.95–2.55	2.60–2.65	2.7–3.10
Water absorption (%)	3.0–20.0	3–15	0.10–0.40	0.05–0.40
Porosity (%)	10.0–35.0	5–30	0.20–1.00	0.15–1.00
Compressive strength (dry) (MPa)	25–100	30–60/25–55[a]	175–350	225–400[b]
Compressive strength (wet) (MPa)	15–90	20–50/20–45[a]	150–300	200–350[b]
Flexural strength (dry) (MPa)	3–10	3–12/2–10[a]	10–30	50–90
Flexural strength (wet) (MPa)	2–8	2–10/1–8[a]	8–25	50–90

[a] Normal/parallel, relative to loading direction.

[b] Small data pool, representivity not certain.

Data abstracted from Hunt, B., 2003. Introduction & limestone. Geol. Today 19 (4), 151–156; Hunt, B., 2005. Granite. Geol. Today 21 (3), 110–116; Hunt, B., 2006. Slate. Geol. Today 22 (1), 33–40; Hunt, B., 2008. Sandstone. Geol. Today 24 (1), 33–38.

porous. The coarser the arrangement, and the higher the constituent proportion of shell fragment to binder, the lesser the volume proportion of cementing matrix and the stronger the stone (Leary, 1983). This microporous character of limestones dictates that they retain water longer than sandstones, and as a result they are in general more prone to decay, especially in an urban environment.

12.3.3 Igneous

Igneous rocks are formed by the cooling and solidification of volcanic magma, which is rich in silicate minerals, for example, melted sand (Siegesmund and Török, 2011). In the UK stone industry, most igneous stone is colloquially referred to as 'granite'; however, much basaltic stone is referred to as 'whin', easy to distinguish by their black colour. Granites and other igneous rocks are essentially non-porous; typically they exhibit porosities less than 1.5% (Urquhart and Artis-Young, 2008). They are impervious, and therefore exhibit incredible durability in response to the agents of decay (typically mobilised by water). The familiar example is the whin kerbing which outlasts the road, reuse after reuse, time after time.

Granites comprise interlocking crystals of quartz, feldspar and mica. The crystals of the respective constituents are visible to the eye. Granites are classified according to the chief mineral other than quartz or feldspar (eg, biotite-granite) (Blyth and De Freitas, 1984). Whinstone comprises fine-grained interlocking crystals typically very dark in colour, of mafic mineralogy (Hyslop et al., 2006). Historically, igneous stones were mainly used in random or squared rubble configurations, owing to their difficulty of working; latterly power-assisted extraction, cutting and dressing techniques have permitted fine ashlar work using igneous dimension stone.

Owing to the impervious and non-porous nature of granite and other igneous stone (Table 12.2), decay issues tend to be attributable to initial poor selection (Hunt, 2005), bad detailing and mortar incompatibility, rather than as the result of weathering processes in service. Igneous building stones are extremely durable.

12.3.4 Metamorphic

Slate is the primary metamorphic building stone used in the United Kingdom. Its primary application is as a roof cladding material; however, certain deposits (a prime example being Cumbrian slate) exhibit thicker splitting planes, and have therefore been employed as a structural building stone, imparting a vernacular form of construction. Much is asked of slate in service; as a roofing material it bears the brunt of the agents of decay.

Slate comprises the most durable of minerals. Whereas in a sandstone, for example, the inert quartz grains are cemented together by a mineral binder, in the case of slate, these grains are fused together. Slate is a 'metamorphosed' mudrock (Hunt, 2006), and is formed by the compression of sediments under high pressure. The 'squashing' of the individual grains dictates that a natural splitting plane is formed perpendicular to the direction of stress. As with the igneous building stones, it is in essence non-porous (Table 12.2), and is impervious, commanding incredible durability.

The value of slate is determined by the uniformity of splitting plane, and the absence of impurities which can quickly decay, leaving pitting in the parent slate. Its durability as a roofing material is determined by strength and water absorption characteristics. The durability of slate as a roof cladding material is unrivalled by clay or concrete tiles, which are inferior in service and lifespan (Architects' Journal, 1987). A naturally slated roof should exceed at least 100 years' performance (Harrison, 2005). The principal factor militating against its use, as with the other natural building stones, has been initial cost. However, the UK slate industry has responded to this shortsighted view, and cost between natural slate and other roofing materials is now very competitive (Hunt, 2006). The advent of standard slate sizes has buried the vernacular diminishing courses of random widths. The environmental consequence of this is reflected in the high wastage associated with the primary slate product.

The requirements for new building stone units (for structural and non-structural purposes) are specified in BS EN 771-6:2005 (BSI, 2011). Whilst the British Standard may guarantee initial characteristics and performance of the stone, the testing does not describe its long-term durability in service (Urquhart and Artis-Young, 2008). Architectural detailing, compatibility of mortar and effectiveness of maintenance (eg, rainwater ironmongery) are prevalent.

12.3.5 Durability of stone

The durability of any material can only be considered rationally when set against the context in which it must function (Turkington, 1995). Natural stone as a building material has proved its durability through the wealth of heritage structures enduring weathering over hundreds of years. Schaffer (1932) defined building stone ‘durability’ as longevity of performance against decay processes. Decay of natural stone is a complex phenomenon, the subject of much research, and in so doing has perhaps received bad publicity. In the context of stonework, the main driver of decay is water, which provides the engine by which the agents of decay are mobilised.

Industry/trade-orientated literature tends to promote a widely held view of frost attack as being the primary source of stone decay (eg, Brick Development Association, 2005, 2006). Certainly, the wet, windy climate of the United Kingdom (especially in the north and west) with a vast number of fluctuations above/below 0°C (Met Office, 2014) would at face value seem to support this view. This is now acknowledged as a misconception, yet the view still persists. Frost damage only occurs in areas of porous material which are very wet and frequently frozen. Water availability for frost attack can be absorbed through rainfall, although Price (1975) noted this to be nominal between the eaves and ground level in buildings within the United Kingdom.

However, frost attack is not a major source of decay in masonry structures in the United Kingdom (Clayton, 1999), largely because the conditions necessary for severe degradation are seldom realised (Price, 1975). Frost attack is more associated with roofing fabric on buildings, and flat concrete elements forming a large catchment for water. The number of cycles, rather than the extreme of the low temperature, are the relevant aspect to frost damage of porous materials. The temperature at which water in pores freezes is variable and cannot be inferred from ambient temperature alone (Benavente et al., 2013).

Where frost attack is misrepresented in prevalence as an agent of decay in masonry structures probably due to its ease of being intuitively grasped, salt decay is often disguised behind ‘frost attack’ or ‘wind scour’. Salt crystallisation in pores is the most prevalent aspect of stone decay in the United Kingdom (Price, 1975; Price and Doehne, 2010; Clayton, 1999). Salts in porous masonries may include chlorides, nitrates, and especially aggressive are sulphates. Salt weathering is similar in mechanism to the damage caused by ice formation in pores; the pore walls are stressed by salt nucleation and growth within the physical constrictions of the solid matrix (Auger et al., 2003). As with frost attack, resistance of the porous material is governed by the microstructure of the stone (eg, Pavia, 1999). The number of crystallisation cycles is again the prevailing aspect. Most of the crystallisation pressures reached in the salt-weathering studies by Auger et al. (2003) exceeded the tensile strength of the stone. It was also found that both fine- and coarse-pored stones are liable to salt damage under the right conditions. Fig. 12.4 demonstrates extensive salt decay in response to accelerated wetting and drying cycles.

12.3.6 Moisture movement through masonry materials

In view of the significance of moisture for the durability of masonry materials, the designer must ensure that it is effectively dealt with in practice. Effective rainwater management largely addresses the brunt of the foregoing decay agents, controlling the engine by which they are mobilised: Deflect the load away from the building fabric. However, wind-driven rain, rising damp and internally released vapour are inevitable moisture loads on masonry structures. This moisture must be recharged to the outer environment, in as sensitive a way as possible: The point at which water evaporates

Fig. 12.4 Bothwell Castle (Scotland, UK). Thirteenth-century round tower (south), extensive decay. Temporary stabilisation work evident.

is the point at which any soluble salts will precipitate and damage the stone (Wiggins and Klemm, 2014).

Breathable internal finishes, prime examples being lime plaster finished with limewash, are important in permitting the internally released moisture to traverse the walls. Modern sheet materials such as gypsum plasterboard retard such egress, and are therefore only suited to cavity construction, and inappropriate for solid wall.

Moisture movement through building materials is a common feature to both modern and heritage structures, and it holds significance for the durability of both (eg, Hens, 2011). Modern building designs typically focus on impermeable, highly engineered materials and details, designed to keep water out. Such structures tend to be less tolerant of design and/or construction defects (Dunbar-Nasmith, 1997; Forsyth, 2008) than old buildings, which adopt a more natural use of construction materials in their designs. Sources of moisture loading on buildings can include rising damp (particularly for old buildings), metabolic vapour release from occupancy, steam from cooking, bathing and so on. This vapour must be allowed to traverse the walls without entrapment and consequent condensation.

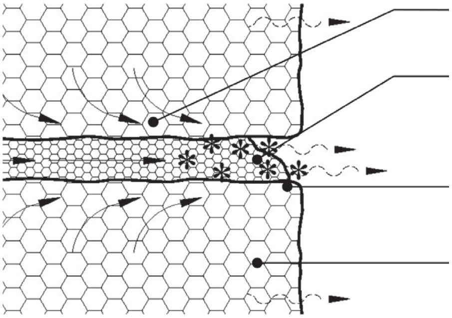
Regarding the external moisture loadings, the bedding and pointing mortar is key to a durable response to moisture, in addition to any surface coatings which may be present. By their macrotexture they deflect incident rainwater, and by their microstructure they draw water out of the fabric of the wall, and recharge to the outer environment. The correct specification of the right stone with the right mortar for a given context is central to promoting the durability of masonry structures.

12.3.7 Sister material: mortar for stone

Natural stone as a durable building material has received some bad publicity, through the appeals for and public prominence of conservation work on stonebuilt heritage. The irony can be appreciated, given the demonstration of longevity in the very structures being conserved. A significant proportion of this repair work is directly attributable to previous interventions using incompatible materials in repointing and surface repair.

Sedimentary dimension stones are porous (Table 12.2). Sections 12.3.5 and 12.3.6 introduced the significance of water for the decay of porous masonry materials. Sedimentary building stones (sandstone, limestone) are therefore susceptible to the agents of decay. Nevertheless, heritage structures constructed of both of these stones exhibit remarkable longevity: Sedimentary building stones can be highly durable, and can be specified for new work with confidence.

The Society for the Protection of Ancient Buildings (SPAB) observed that the lime mortar in the stonework joints wicked absorbed water out of the masonry units (SPAB, 1979). The subsequent evaporation therefore is most concentrated in the lime joints, which receive the brunt of the degradation processes which then unfold. The underlying principles are reviewed in Wiggins and Klemm (2014) and Klemm and Wiggins (2015), and may be summarised as the poulticing of water (and soluble salts) away from macroporous stone into the microporous lime mortar, whose pore size distribution is finely tuned to impart this unique phenomenon, itself a remarkable measure of durability (Fig. 12.5). Non-porous stones (igneous, metamorphic) also benefit from being constructed with lime



Migration and evaporation of water through the lime mortar joint

Concentration of soluble salt precipitation, manifesting in the sacrificial weathering of lime mortar

When exhausted, the mortar becomes friable and is then raked out and replaced, refreshing the protection for the next service life

Typical coarse-pored sedimentary dimension stone, notably sandstone and limestone. Historic bricks similar.

Fig. 12.5 Bed-joint cross-section showing sacrificial weathering of lime mortar and subsequent repointing.

mortar; however, the benefits are more associated with mechanical compatibility criteria, which of course have knock-on implications for moisture-related processes.

It is worth noting that those advocating the use of modern masonry as a durable construction material capable of spanning many centuries invariably draw on heritage examples, which enjoy the sacrificial protection offered by the lime mortar, but seldom credit where it is most deserved. By contrast, a material as durable as natural stone can in a comparatively short timeframe deteriorate beyond salvage with the application of incompatible materials. Cement mortar repointing and surface repair attempts within the last century have markedly scarred stonebuilt heritage in the United Kingdom. Addressing this issue forms the vast proportion of current repair work to heritage structures. A parallel is drawn between mortar-stone and bimetallic corrosion of galvanised-stainless steels.

With the topic of compatibility in mind, the contextual use of stone should be examined: Where the optimal durability associated with the sacrificial protection of lime mortar is sought, the designer should be aware that the stonework cannot then be designed to resist significant flexure across the mortar joints (as is modern practice, through the high strength of cement mortar). Lateral loads are instead typically resisted by the static resistance offered by gravity loads and restraints (Curtin et al., 2006). This consideration is held in the same vein as the low-strength mortars endorsed by the Building Research Establishment (BRE) (Key, 2009) to allow for future reclamation of the masonry units. Further advantages of using lime mortars include joint deformability, negating movement joint provision in solid wall construction. The use of cement mortar with natural stone is not recommended. A range of lime mortars are available to meet the requirements of building stone in both heritage and modern contexts (see Urquhart and Artis-Young, 2008).

12.3.8 Repairability of stonebuilt structures

Traditionally constructed structural masonry is eminently repairable. Again, one can look to stonebuilt heritage for testament to this attribute. With increasing emphasis being placed on longer and longer design service lives for buildings (eg, the BRE recently suggested 200 years would be more appropriate; BRE, 2006), repairability becomes an important measure of building sustainability.

Traditional structural masonry represents an adept yet honest exploitation of stone as a building material, designed wholly in compression. Stone masonry is extremely strong in compression, yet its ability to support tensile/flexural loadings is negligible (Heyman, 1995). Playing to the material strength and designing out its limitations, the masonry arch aptly serves as example. Capable of sustaining high loads over a long lifespan, it works wholly in compression provided its geometry is maintained and its constituent material does not decay. A blunt way of expressing this is the more honest the exploitation of material properties, the less can go wrong with the structure.

Referring again to the sacrificial weathering of lime mortar jointing in traditional masonry, stonebuilt structures require very low maintenance over their long lifetimes (Powys, 1929; Hume, 2007; SPAB, 2002). The bedding/pointing mortar draws the water (and soluble salts) away from the masonry units and concentrates the agents of decay within the lime. This then receives the bulk of the harmful salt precipitation cycling, eventually weathering away (Fig. 12.5). When exhausted, the mortar can readily be raked out and replaced, refreshing the sacrificial protection outlined in Section 12.3.7. Provided the correct material is employed in successive repointing exercises, this process could be maintained indefinitely. The reader is directed to SPAB (2002) for a practical account and specification guidance.

12.4 Stone performance

12.4.1 Introduction

Stone is a natural material. However, in the case of bricks, for example, where the natural constituent (clay) has to be fired in a kiln (a process representing some 85% of its EE), for stone this work is done in nature. Stone is 'pre-made'. However, it must be accessed, extracted and then worked before it can be used as dimension stone. These aspects necessarily have environmental impacts and an objective means of appraising these (comparing with other materials) is afforded by assessing the energy and carbon dioxide impacts of each stage, which are 'embodied' in the final product.

Embodied energy (*EE*) is the amount of energy required to extract, process and transport a material to the point of use or application. *Embodied carbon* (*EC*) is the amount of CO_2 associated with this procedure, released to the environment. It includes the CO_2 released by fuels, but also accounts for the CO_2 driven off, for example, when burning limestone to make lime or cement. Note, this burning process is not applicable in the extraction or production of dimension stones.

The measure of EE and carbon is most practically appraised at 'cradle-to-gate' (C-G) level; that is, the energy and carbon required to extract, process and deliver the material to the point of sale. In the context of natural stone this would stem from the quarry to the gates of the processing facility. Of course the actual energy or carbon incurred includes the transport of the final product to the point of application or use ('cradle-to-site' —C-S), and any handling thereafter, although this varies widely and would be the focus of a bespoke life-cycle assessment (LCA) for a particular project. A third accounting level is 'cradle-to-grave', which accounts for the final deconstruction and disposal of the product.

EE and carbon of dimension stone accounts for not only the finished product, but also the waste associated in developing the product. This impact-apportioning is termed *allocation*, and goes to the primary product (ie, dimension stone). The EE/EC figures in the following tables should be viewed whilst bearing in mind the fact that UK quarries produce practically zero waste to landfill. This contention becomes particularly prevalent in the case of slate, which may produce some 90% wastage in developing the dimension slates, which is necessarily reflected in its comparatively high figures. Stone off-cuts, chippings, aggregates and so on are all marketable by-products; however, they are allocated none of the EE/EC impact: The primary product 'takes the hit'.

12.4.2 EE and carbon footprint of stone

Much of the EE/EC footprint of natural dimension stone is due to the processing of the blocks after extraction, and there remains ample scope to reduce the current figures further: Some 90–95% of the EC associated with dimension sandstone was shown by Crishna et al. (2011) as being due to the processing alone. Renewably powered processing facilities could drive stone's EE/EC footprint down to incredibly low levels.

Natural stone has a low EE and carbon footprint (Tables 12.3 and 12.4). However, objective comparison between materials of different build contexts is difficult: Masonry materials tend to be used in heavy constructions, whilst the materials associated with higher EE/EC footprints tend to be lighter. This difficulty is resolved by the LCA for the structures respectively, although a feel for comparative purposes can be gained by comparing constructions in the BRE's Green Guide (Anderson et al., 2009). The context of use issue is well represented in slate; whilst there are exceptions in which it is used in structural walling, its predominant application is as a roofing material. Therefore, a unit area assessment of the EE/EC footprint would be in order.

Table 12.3 **Embodied carbon footprint of selected natural stones available in published literature**

Embodied carbon of dimension stone (cradle-gate)				
Source	**Sandstone ($kgCO_2/t$)**	**Limestone ($kgCO_2/t$)**	**Granite ($kgCO_2/t$)**	**Slate ($kgCO_2/t$)**
Hammond and Jones (2008)[a]	60	90	70	7–63
Crishna et al. (2011)	64	–	93	232
University of Tennessee[b]	–	105	62	28

[a] Hammond & Jones' Inventory of Carbon and Energy (ICE).

[b] University of Tennessee (2008a–c, 2009).

Table 12.4 Embodied energy footprint of selected natural stones available in published literature

Embodied energy of dimension stone (cradle-gate)				
Source	**Sandstone (MJ/t)**	**Limestone (MJ/t)**	**Granite (MJ/t)**	**Slate (MJ/t)**
Hammond & Jones ICE	1000(?)	410	4100	30–1400
University of Tennessee	[a]	964	5908	208

[a] NB: Processing data was unavailable from the University of Tennessee; however, the quarrying cost was lower than limestone, and processing/dressing techniques to produce dimension stone are quite comparable between sandstone and limestone.

The data pool for the EE and carbon environmental impact of natural dimension stone was noted by the source authors to be small, leading to the relatively high deviation observed. The figures quoted in Tables 12.3 and 12.4 are largely composed of the energy (and related CO_2 impact) in the processing of the quarried stone. The EE figures do not appear to accord well with the EC figures, across the few sources available. The figures indicated consider the ‘waste’ associated with the final product (dimension stone), which is very high for slate. However, Crishna et al. (2011) observe that, owing to the zero waste in practice arising from stone quarries, considering the environmental footprint of stone on the total material produced (including secondary products of rubble and hardcore, etc.), slate can be demonstrated to have lesser carbon impact than the other building stones.

Interpreting the data generally, taking average values across the different sources, it is clear that natural stone exhibits a low environmental footprint when compared with other typical building materials (Table 12.5).

The mortar with which the stone is laid represents a major constituent in the bulk of a stonework wall, particularly where the masonry units are small (demanding a higher frequency of joints, as in brickwork). Traditionally, stone masonry was laid with lime mortar. As with natural stone, the data pool for lime mortars is correspondingly small, and some contention is to be found in the literature as to its environmental footprint relative to cement mortar (eg, Hammond and Jones, 2008; De Vekey, 2005; Reddy and Jagadish, 2003).

The lower burning temperature of lime does not necessarily or directly constitute an environmental saving, owing to the typically longer burning durations. However, lime mortars feature a carbonation set, which reabsorbs an extent of the CO_2 driven off when the limestone is burned. This is especially so for putty limes, which set entirely by carbonation and fully reabsorb the displaced CO_2; the low end of the natural hydraulic lime (NHL) (eg, NHL 2) similarly exhibit a high degree of carbonation. Little difference is observed between NHL 3.5 and NHL 5 mortars in terms of their respective proportion of carbonation set (Hughes and Swann, 1998). The formation of the compound which demands the carbonation set (portlandite) is strongly influenced by the phase chemistry, but in general terms the lower the burning temperature, the higher the proportion of portlandite formed and available for carbonation. For the

Table 12.5 **Embodied carbon footprint of selected building materials abstracted from Hammond & Jones ICE**

Material	Embodied carbon (kgCO_2/t)
Aluminium (general)	8240
Cladding panel (fibre-cement, colour-coated)	1280
Concrete (RC28/35)	139
Conc. block: 8 MPa	59
Conc. block: 13 MPa	100
Conc. block: AAC	240–375
Brick: Facing	520
Brick: General	230
Glass (toughened)	1350
Steel: General	1460
Steel: Plate	1660
Steel: Section	1530
Timber: Hardwood	470
Timber: Softwood	450
Sandstone[a]	62
Limestone[a]	98
Granite[a]	75
Slate[a]	96

Note: All UK values.

[a] Mean average from Table 12.3.

more hydraulic mortars (NHL 3.5, NHL 5 and cements), the mineralogy and higher burning temperatures lead to smaller proportion of portlandite formed, and consequent carbonation is small if at all. Carbonation is a complex process related to available portlandite, diffusivity of the pore matrix and environmental conditions.

Given the foregoing, support is made for the use of the weaker, high-carbonation limes. Reddy and Jagadish (2003) observe that lime:pozzolana mortars have the lowest environmental impact of all the building mortars they studied, including cementitious. Table 12.6 abstracts from Hammond & Jones Inventory of Carbon and Energy (ICE) the data for cement mortar, and the closest available mixture to represent a lime mortar.

Table 12.6 **Environmental impacts of building mortars abstracted from Hammond & Jones ICE**

Characteristic	Lime (L:A)	Gauged lime (C:L:A) 1:2:9	Cement (C:A) (1:3)
Embodied energy (MJ/t)	–	1030	1330
Embodied carbon (kgCO_2/t)	–	145	208

Data from Hammond & Jones ICE.

Appraising the difference between cradle-to-gate and cradle-to-site across the stones is made difficult owing to the single data source available. However, Table 12.7 allows comparison between stone sourced nationally and imported stone from abroad. The imported figures quoted were derived by Crishna et al. (2011) assuming the shortest/most practicable route, and that processing was undertaken in the country of origin.

The environmental footprint of stone increases many times over the further away the source is located. Clear suppport for the use of indigenous stone is demonstrated in EE and carbon terms, aside from the less-quantifiable sustainability benefits of promoting local rural employment, and safeguarding local distinctiveness.

12.4.3 Whole-life cost (carbon, energy, financial)

Whole-life cost (WLC) is the joined-up appraisal of initial cost, operational cost, maintenance burden and eventual deconstruction of a given building. It adopts a cradle-grave approach, and represents a framework for appraising the costs of a structure in EE/carbon and financial terms separately.

Masonry structures are extremely durable and require very little maintenance over their lifetimes. Such maintenance typically involves periodic repointing, surface repair, and stone replacements of individual units as required. As with buildings of any construction form, the roof requires routine maintenance. Traditionally constructed roofs clad in natural slate are widely known to outlast other roofing systems; slated roofs as with stonebuilt structures are durable and eminently repairable.

Operational costs in buildings are largely dependent on how they are used by the inhabitants. In terms of the internal fixtures (heating and lighting systems, etc.), stonebuilt structures can be outfitted in the same way as other forms of construction, historic and new-build alike. Space heating is a major operational cost in domestic dwellings in the United Kingdom, and to a lesser extent air conditioning. At least half of the environmental footprint of a typical dwelling over at least 50 years' service life is due to operational user energy consumption. Such relationships necessarily vary according to the build cost in environmental terms, the efficiency of the building in service and the lifespan of the structure itself.

Table 12.7 Embodied carbon footprint of imported natural stones available in published literature

Embodied carbon of imported dimension stone (cradle-site)				
Country of origin	**Sandstone ($kgCO_2/t$)**	**Limestone ($kgCO_2/t$)**	**Granite ($kgCO_2/t$)**	**Slate ($kgCO_2/t$)**
UK	77.30	–	158.00	297.40
Spain	134	–	161	318
Poland	–	–	188	–
India	312.30	–	336.50	–
China	504.00	–	415.50	568.10

Data from Crishna, N., Banfill, P., Goodsir, S., 2011. Embodied energy and CO_2 in UK dimension stone. Resour. Conserv. Recycl. 55, 1265–1273.

The temperature difference between internal and external environments together with the thermal characteristics of the envelope cladding dictate the operational space heating demand for a given building configuration. Passive gains from natural lighting and thermal mass also influence the operational efficiency of masonry structures (Halliday, 2009). Traditionally constructed buildings if appropriately insulated and outfitted with efficient heating systems, etc., can outperform new buildings over the life of the building when EE/EC is considered (Crishna et al., 2011). To compare different building materials, it is convenient to set the building configuration and temperature gradient constant, and compare U-values of different claddings. However, research has found that with the black radiation emanating from stone, coupled with the slow changes in internal temperature relative to the external environment, on the whole lower temperatures are found tolerable in terms of thermal comfort, relative to other forms of construction (Hutton and Roston, 1997; Athienitis and Santamouris, 2002).

The BRE's Green Guide collates a breadth of building materials and building configurations, to allow this cross-comparison. It contains inherent idealisations, notably the 60-year design life assumed. The short-term design life of buildings is now recognised as unsustainable, and a recent drive for longer design lives proposed 200 years (BRE, 2006). The Green Guide underplays the sustainability credentials of natural stone (Yates and Bourke, 2005).

Such idealisations serve to disadvantage stone against other building materials. Value in stone can only be fairly demonstrated on a WLC basis. The (relatively high) initial cost, if not considered against the full life term or the very low maintenance costs over that long life, makes stone appear very expensive in a short-term view. A WLC appraisal also recognises the reclaimability of natural stone, which retains intrinsic value, and for which there is a strong market demand (Wilson, 2005).

12.4.4 Thermal performance of stonebuilt structures

Stone as a building material has high-density, high-thermal capacity but low thermal resistivity: It is generally a poor insulator unless used in extensive thicknesses, as has historically been the method adopted. Thinner configurations can achieve good insulating properties if supplemented by insulation, and both solid wall and cavity stone masonries are suitable to receive such measures.

Stone yields very high thermal mass, in both solid wall and cavity form. Thermal mass is the ability of the material to absorb, store and release heat energy, and in doing so self-regulate the internal environment from temperature fluctuations. Heat is absorbed and stored, until the surface is exposed to cooler temperature and the heat then gradually migrates back out, evening out fluctuations in temperature. This cycle typically occurs over a 24-h period, and occurs both in summer and winter.

The introduction of cavity fill as insulation ‘jeopordises durability’ (Key, 2009). It pays to remember that the advent of the cavity in masonry construction was to break capillary continuity across the wall, to cater for the wet UK climate: Cavity fill serves to bridge the cavity. Traditional solid masonry walls of ample thickness stay dry on the inside, as absorbed moisture from wind-driven rain is wicked outwards and recharged to the external environment by the lime mortar and general breathability of the stonework. In the context of a retrofitted cavity fill in an otherwise cement-based brick masonry construction, breathability is significantly retarded, and absorbed moisture struggles to escape. Moisture retention in masonry serves to reduce its insulating properties, promotes mould germination and in general upsets the healthy operation of the building.

Solid wall stone masonry inherently excels at achieving high degrees of air-tightness of the walls. Current building regulations (eg, HM Government, 2013) allow new-build construction to pass at an equivalent of a 20-pence-piece hole through an impermeable fabric every square metre. Irrespective of the insulating characteristics of the walling fabric, heat moves faster by convection than diffusion, and the quoted U-value will not be realised in practice whilst heat can escape through leakages. Furthermore, cavity construction requires technical mechanical tying, restraints, etc., which serve to cancel out the face-value saving on bulk masonry material which would be required of a solid wall configuration. Solid wall is synonymous with optimal durability, and features a higher degree of adaptability when compared to cavity construction.

An increasingly prevalent future trend is the need to design buildings for hot summers in addition to cold winters; the building should shelter against both extremes, even in the United Kingdom. The passive self-regulating environments offered by structures of high thermal mass are being recognised as key to meeting this demand (ARUP Research & Development, 2005). Stone masonry is an ideal medium as part of this fabric energy storage system. However, thermal mass needs to be accessible by the internal environment to engage. The Concrete Centre (2006) compared timber-framed construction and cavity masonry, in which the heavyweight masonry achieved the same U-value as the timber framed, and six times the thermal mass admittance.

12.5 Sustainable use of natural stone in construction

12.5.1 Reclaimability of masonry units

New masonry units are specified and supplied in accordance with EN conformity criteria. An implication of reclaimed masonry units is that durability cannot be guaranteed (Brick Development Association, 2014). Depending on the intended use, this may or may not present specification issues; where used for a non-structural cladding, guaranteed mechanical characteristics may not be relevant. Where used in a traditional structural context of low stress and robust geometrical stability, mechanical material characteristics of the stone are of low relevance (Heyman, 1995).

Reclaimed masonry units are currently synonymous with high cost because of their small, niche heritage market and high labour intensity involved in the reclamation process. Reclamation is typically a hand-process, by hammer and chisel. However, an emerging measure of sustainability, as endorsed by the BRE's Green Guide (Anderson et al., 2009) is that structures should be designed for deconstruction and reuse at the end of their service life.

The majority of modern bricks in demolition projects go to landfill (Addis, 2006). The high-strength cementitious mortars employed preclude effective reclamation.

Modern brickwork is produced with increasing emphasis on perforations, to minimise material volume, increase the efficiency of the firing process and improve thermal properties (Torgal and Jalali, 2011); perforations further preclude effective reclamation. BRE's guidance set down in (Key, 2009) endorses the use of ‘low-strength’ mortars which allows for future reclamation. Traditional lime mortar meets this requirement, and reuse of stone and brick is a long-established tradition.

12.5.2 Stone in a modern context

Stone masonry, like brickwork, is a very versatile building medium. Used for thousands of years in solid form, stone also readily lends itself to different modern forms of construction. In a modern context, stone generally either forms part of a composite load-bearing wall (eg, as the outer leaf or facing leaf of an otherwise brick/block masonry wall), or indeed solely as a decorative cladding material. In the former, stonework supports its own weight and shares lateral loading; in the latter, the stonework carries its self-weight only. The selection of stone in either modern context is initially for its aesthetic appeal, but this decision can be shown to represent savings on environmental impact.

As a decorative cladding material, little thickness is required. Whilst on face value this reduces the mass of primary product used, thinner products have markedly increased processing requirements, forcing high degrees of primary wastage (Wilson, 2005). Such thin decorative claddings are generally the subject of bespoke design. As a rule, practical minimum thicknesses for normal cladding contexts are 100 mm for single-storey (3 m) ashlar; 150 mm for two-storey (6 m), and 150 mm minimum for rubble-faced masonry (BSI, 2005). Reference to Urquhart and Artis-Young (2008) is recommended for guidance on the specification and detailing of natural stone as a cladding material in modern construction.

Traditionally built stone structures neither feature nor need movement joints. Opinion is not unanimous within the engineering community as to the precise reason(s); however, the two main aspects are agreed to be: (a) the deformability of the lime mortar joint, which acts as a damper between masonry units to absorb any expansion on a local scale without cracking; (b) the high thermal mass of thick stone walls, slowing down the temperature changes. Porous stone also experiences dimension changes in response to moisture content, and the lime mortar certainly allows for each stone unit to swell/contract without cracking. In the modern (thinner) construction contexts, formal movement jointing is required in addition to mechanical restraint systems.

Appropriately selected and detailed, taking account of the practical thickness minima, the use of natural stone should present no more difficulty than brickwork cladding: restraint, ventilation and movement provision are required as a matter of course. On this basis, it is interesting to observe the savings in environmental impact with the replacement of a brick outer leaf with natural stone.

In view of Table 12.8, supposing a standard common brick outer leaf with some 20-mm cement render were to be replaced with sandstone cladding (Fig. 12.6A), a remarkable 75% saving in EC can be demonstrated. Opting for sandstone cladding in place of facing brick would represent an EC saving of some 90%. In a build context

Table 12.8 Overview of cladding masonries' environmental impact

Material	Density[a] (Mg/m^3)	Thickness (mm)	Tonnage (t/m^2)	Embodied carbon (kgCO$_2$/t)	Embodied carbon (kg/m^2)
Common brick	2.14[b]	100	0.21	230	49
20-mm Cement render	1.79	20	0.04	208	7
Common brick, rendered	–	–	–	–	57
Facing brick	2.14[b]	100	0.21	520	111
Sandstone[c]	2.18	100	0.22	62	13
Limestone[c]	2.25	100	0.23	98	22
Granite[c]	2.63	100	0.26	75	20
Slate[c]	2.90	150[d]	0.44	96	42

[a] Densities are typical values, mean average from Table 12.2.
[b] Approximation abstracted from mean value across the mass of brickwork outlined in Cobb (2015), assuming standard vertical perforations are filled with mortar.
[c] Embodied carbon values are mean averages from Table 12.3.
[d] Slate included as minimum thickness 150 mm as used in rubble form, especially in Cumbrian area of the United Kingdom.

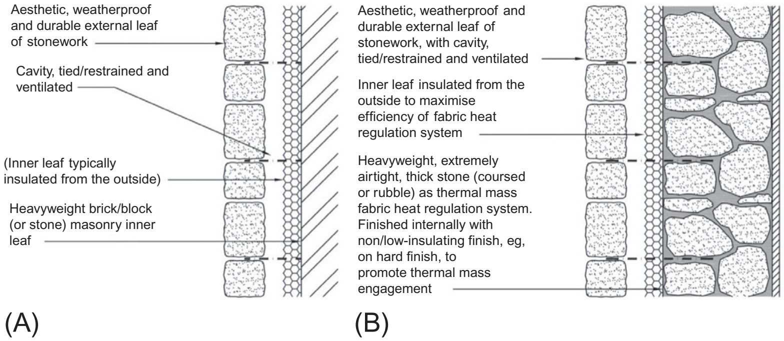


Fig. 12.6 (A) Stone outer leaf as part of a composite masonry wall. (B) Stone outer leaf with thick inner leaf as thermal mass fabric heat-regulation system.

with, say, a 100-mm stone outer leaf, coupled with a heavyweight thicker inner leaf separated by a cavity, would allow the aesthetic appeal and environmental benefit of the outer leaf to be combined with the thermal mass ‘heat-store’ capabilities of the insulated inner leaf to be best realised (Fig. 12.6B).

As stated, the use of natural stone in construction does not have to be limited on aesthetic grounds: Environmental impact savings can equally be made to the inner leaf: replacing, for example, a 150-mm-thick concrete block with equal thickness of stone not only lessens the carbon footprint of the wall, but also markedly increases its mechanical properties (compare 8–13 MPa concrete blocks with Table 12.2). The beauty and character, in addition to durability and mechanical properties associated with natural stone, come as welcome by-products when measured in terms of environmental impact.

12.5.3 Sociological sustainability of stonebuilt structures

The way in which people experience the built environment is complex (eg, Lawrence, 1987). Affectionate association with buildings is tied to character and distinctiveness, which are much deeper attributes than functional provision. Sense of place is now widely recognised as being central to a fulfilling environment (The Scottish Government, 2013). Sense of place goes far beyond utilitarian function. Identity is important, and many look to the past. The prominent endurance of stonebuilt heritage can offer a sense of continuity in a changing world (English Heritage, 2011). The durability offered by stone as a building medium underpins this provision.

Natural stone is synonymous with ‘loveability’ of buildings, a ‘dimension of home’ (Hayward, 1975). The effort and affection of the people who worked on them can be appreciated (Dunbar-Nasmith, 1997). Stone improves in attractiveness with exposure and time. A receptive host for natural/organic microorganisms, they develop a patina of age. Natural variations and random imperfections help create a beauty which cannot be rivalled by modern emulations. It is the material of choice when prestige requires display.

Difficult to quantify, the ‘amenity potential’ of the sustainability three-pillars is often avoided, and sustainability is reduced in practice to financial and environmental cost. A qualitative measure of amenity value imparted by natural stone in building can surely be found in the local affection of communities to their heritage structures. Stonebuilt heritage is also often associated with sociological sustainability, reasoned by the BRE (Plimmer et al., 2008) as being due to the fact that historically, dwellings were built around community hubs and local workplaces, encouraging local community engagement. Bland system-builds have diluted local distinctiveness.

When coal mining ceased across the breadth of the country, many rural communities which had been built around that employment were decimated, and former mining areas are now often stigmatised, remote from places of work. Stone quarrying and processing remains one of the few profitable rural endeavours (Yates and Bourke, 2005), and is not driven by industrial boom. Dunbar-Nasmith (1997) observed that it is senseless for a government on the one hand to support trade employment schemes, yet on the other to continue to endorse buildings designed to employ the minimum amount of skilled labour in their construction. Machinery and technology both have the potential to give employment, as well as to take it away (McDowell and McCormick, 1999). Sustainable development then must reject overexploitation of both the environment and of people. Revival of the use of stone in building, particularly in a structural context, has application in both the developed and the developing parts of the world, especially the latter where labour is comparatively inexpensive.

Conservation of heritage structures will always offer a degree of work for the stone industry. However, investment in the revival of the skilled craft of stonemasonry, by way of creating increased, long-term demand, has to stem from confident selection of natural stone in both the heritage and modern contexts. Investing in the use of natural stone truly addresses the three pillars of sustainability, not least the sociological/amenity criterion: stable employment on the one hand, coupled with a restoration of local distinctiveness on the other through the as-built product.

12.5.4 Future trends

The drive towards a lower carbon footprint of construction in the United Kingdom is coupled with a drive for increased operational efficiency of buildings. Heavyweight construction is now recognised as being key to designing sustainable buildings which are tolerant of the changing climate. Middle ground between these aspects is to be found with the modern contexts indicated in Fig. 12.6.

Masonry in general represents an ideal sustainable building form, but natural stone outperforms brickwork on environmental measures. The firing process alone is responsible for approximately 85% of a fired brick's EE. Consequently, developing measures on the brickwork end of the masonry spectrum appear to be focussed on increasing the perforations in bricks, to minimise the clay demand and increase efficiency of firing. Unfired or low-temperature-fired materials are also emerging onto the masonry market. The knock-on implications of their use are a significant compromise in mechanical performance, impaired durability in service, and importantly, the reclaimability of highly perforated units is precluded altogether.

In light of the extensive design lives now recognised as an essential measure of sustainable building, it does not make sense to compromise with a highly perforated masonry unit, when natural stone remains underexploited and has a demonstrable wealth of sustainability credentials. Development in practitioner/specifier-orientated technical guidance has opened the door to employ natural stone in building with confidence.

12.5.5 Additional sources of information

The following texts are recommended for further detail on specific aspects overviewed in this chapter:

- A Geology for Engineers (Blyth and De Freitas, 1984)
- BS EN 771-6:2011 (BSI, 2011)
- BS 8298-1:2010 (BSI, 2010a)
- BS 8298-2:2010 (BSI, 2010b)
- Building Masonry with Lime-Based Bedding Mortars (De Vekey, 2005)
- Compatible Repairs to Stonebuilt Structures (Wiggins and Klemm, 2014)
- Manual for the Design of Plain Masonry in Building Structures to Eurocode 6 (IStructE, 2008)
- Natural Stone Masonry in Modern Scottish Construction: A Guide for Designers and Constructors (Urquhart and Artis-Young, 2008)
- Repointing Stone and Brick Walling (SPAB, 2002)
- Stone in Building: Its Use and Potential Today (Ashurst and Dimes, 1984)
- Structural Masonry Designers' Manual (Curtin et al., 2006)
- Sustainable Masonry Construction (Key, 2009)
- The Stone Skeleton (Heyman, 1995)

The following organisations provided advice on specific aspects overviewed in this chapter:

- BGS
- Building Limes Forum
- SPAB
- Stone Federation Great Britain

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BSI, 2010b. Code of Practice for the Design and Installation of Natural Stone Cladding and Lining. Part 2: Traditional Handset External Cladding. BS 8298-2. British Standards Institution, London.

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Sustainability of compressed earth as a construction material

13

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13.1 Introduction

Majority of publications in construction materials mainly focus on engineering properties with little or no attention on sustainability (Khatib, 2009). Sustainability of a construction material takes into account the long-term environmental, social/cultural, and economic impact on the society. Compressed earth blocks have a record of accomplishment as sustainable from being a natural material with less embodied energy. However, Minke (2000) observed that earth has to be sheltered against rain and frost by roof overhangs, cement, lime or bitumen stabilisation, dampproof courses, appropriate surface coatings, etc. Chemical stabilization and planning of an adequate roof overhang to protect earth wall is very challenging. The cost of roof overhang in the context of modernity can be enormous (Egenti et al., 2013). UNCHS Habitat (1986) observed from field experiences that a majority of the world's earth houses in rural areas suffer from common defects identified as surface erosion, partial crumbling, unhealthy conditions due to constant humidity and hollowed bases. Traditionally, the poor performance of earth wall has left a negative social stigma where even the owners of earth building regard it as an act of poverty and look forward to when progressive sons and daughters (or future generations) will bring a better life in cement buildings. In view of the foregoing, it may be difficult to establish a balance between the three key points of the triangle of sustainability—namely, economic, social-cultural and environmental—in respect of traditional and modern earth construction systems.

In this chapter, the economic, social-cultural and environmental issues relating to earth construction are examined, and a case study is presented on shelled compressed earth block (SCEB), which may balance these three factors as a truly sustainable option.

13.2 Environmental issues

Earth is a natural material with less embodied energy, with minimal emission of CO_2 (Keefe, 2005; Minke, 2006; Glavind, 2009). The addition of cement, lime or bitumen to earth before compression changes the characteristic of earth from its unstable state of strength and volume with changing moisture conditions to a more suitable building material, especially in the tropics (Hammond, 1973, 1991; Ayensu, 1996; Gidigasu, 1993). However, according to Glavind (2009), the production of 1 kg of

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cement generates approximately 0.8–0.9 kg of CO_2 emission. The use of energy, fuel or electricity in cement production process also takes its toll from the limited but much needed energy in the developing countries. An earlier research carried out by Egenti et al. (2013), revealed that there was need for 8–12% cement content for the two earth samples used for the test to be truly durable in the contemporary context. Further analysis in Table 13.1 shows that high cement content is required for earth wall with adequate surface resistance against wear.

The amount of cement calculated in Table 13.1 for the two soil types was high for Aviele laterite soil and outrageous for Ubiane soil, which ranked close with sandcrete blocks; almost one 50-kg bag of cement goes for every 1-m^2 area of wall of an earth building. This raises sustainability issues in earth construction system. The SCEB shown in the green area of Table 13.1 reduced considerably the overall cement content. The amount of cement reduced to 15.5 kg/m^2 area of SCEB wall as against 35.2 kg of Aviele laterite soil, 49.7 kg of Ubiane soil and 52 kg in sand-cement block. The concept of shelled compressed earth construction is discussed later in this chapter.

Soil mining may also raise environment issues if not planned properly. Arbitrary digging of laterite soil has degraded most sites by creating water-logged dishes. Sources of soil should be defined and integrated into the grand plan of the project. Integrating a water reservoir infrastructure, incinerator (for waste treatment and disposal) or basement-to-earth building could provide the volume of soil required for the building.

13.3 Social-cultural issues

The social stigma created by poor performance of earth walls in traditional environments in Africa cannot easily be cleared by a builder's recommendation, but by gradual observation of prototypes over time. This calls for the production of more sustainable earth-based materials that are socially acceptable!

In the contemporary context, the unstabilised earth block or adobe is fast losing its popularity as wall construction material in urban areas. It is therefore an undisputable case of unstabilisation against social rejection. Earth stabilisation for building purposes is a middle path to modernity that is inevitable if earth will become a popular material in the contemporary context. However, a research effort to achieve a wholesome and durable block wall with less cement, lime or bitumen should be a realistic focus of sustainable development efforts.

13.4 Economic issues

Soil can be excavated from the building site thereby eliminating transportation cost. In comparison with other building materials, Minke (2000) observed that even if soil is transported from other construction sites, it is usually much cheaper than other building materials. Earth construction is labour intensive, but can be executed

Table 13.1 **Comparison of soil and cement requirements for durable compressed earth and sand–cement wall**

Description of items	**Aviele laterite soil**		**Ubiane soil**		**Sand–cement**	**Shelled compressed earth block**		
Volume of dry soil required for 1 m^2 of 200 mm thick wall	0.305 mm^2		0.305 mm^2		–	0.305 mm^2		
Mass of dry soil required for 1 m^2 of 200 mm thick wall	362.2 kg		362.2 kg		–	Shell 99.5 kg	Core 177.5 kg	Void 24% (87 kg)
Mass of cement required for 1 m^2 of wall	29 kg	8%	43.5 kg	12%	32 kg	10 kg (10%)	0 kg (0%)	0 0
Mass of cement required for mortar of 1 m^2 area of wall	6.2 kg		6.2 kg		5 kg	5.5 kg		
Mass of cement required for 2 × 1 m^2 plaster	–		–		15 kg	–		
Total amount of cement required	35.2 kg		49.7 kg		52 kg	15.5 kg		

by many unskilled workers with a professional controlling the building process. Thus, self-help construction is encouraged. The analysis made by Guillaud et al. (1985) revealed that a compressed earth brick wall cost 32% less than one made with sand–cement blocks (SCBs), though a reduction in the cost of wall may not impact significantly on the overall cost of the building. The cost of wall constitutes about 30–50% of the overall cost of a building. Most developers will naturally desire high performance at minimum cost. However, adequate stabilisation is required for high performance and weather resistance. In respect of achieving high performance, the analysis of Egenti et al. (2014) in Table 13.1 revealed a high amount of cement with a cost implication.

13.5 Prospects of durability of cement stabilisation

The prospect of durability of earth walls is of utmost importance to social acceptability and the future of earth construction. The durability of earth walls is dependent on how well the forces of erosion on its surface are resisted. The principal mechanism causing erosion of the surface of earth walls is the release of the kinetic energy from raindrops impacting on the surface (Heathcote, 1995). Brooks et al. (2003) corroborated this and further identified a second mechanism, which is the erosion due to concentrated water flow on areas of the wall.

The addition of cement, lime or bitumen to earth before compression changes the characteristic of earth from its unstable state of strength and volume with changing moisture conditions to a more suitable building material in the tropics (Hammond, 1973, 1991; Ayensu, 1996; Gidigasu, 1993). The weather-resistant ability of earth walls is greatly improved when stabilised and compressed (May, 1984; Smith, 1987). The suitability of the previously mentioned materials may be dependent on the composition of the earth as cement is more suitable for an earth with a higher percentage of sand, while lime is more appropriate with a clayey earth as it reacts, though slowly, with clay to form a stable pozzolanic material (Rigassi, 1985). However, the considerable amount of water required for lime and bitumen in the process of production may restrict their use to adobe blocks and not compressed earth blocks that require a small amount of water for good compression (Rigassi, 1985). The availability of these binders could also be a factor for a choice in local application. The presence of iron oxides in lateritic earth allows stabilisation to occur efficiently with impressive hardening effect with little cement. There have been recommendations, in the literature, that 5–8% cement is required for the satisfactory performance of compressed earth walls (Venkararama and Jagadish, 1987; Asamoah-Boadu and Afukaar, 2001).

There is, however, the opinion that the addition of cement to earth raised fundamental sustainability questions against earth construction. The analysis made by Walker and Standards Australia (2002) indicate that clay bricks consume between 1.0 and 7.0 GJ/tonne, of which 80–90% is used in drying and firing; thus replacement of firing with cement stabilisation will reduce the energy consumed in the block production process by at least 50%.

13.6 Sustainability: A focus of modern research

In the middle of the 20th century there was an emergence of environmental issues regarding the importance of sustainable development by constructing environmentally responsible houses. This placed earth construction of modern houses in the focus of modern research.

A number of institutions, governmental organisations, schools, research centres, agencies and individuals have been researching, promoting and developing earth architecture for creation of sustainable shelter in developing countries. They include the International Centre for Earth Construction (CRATerre-EAG) in the School of Architecture, University of Grenoble; UN-Habitat; Centre for the Development of Industry (CDI); UNESCO Chair for Earthen Architecture; International Centre for the Study of the Preservation and Restoration of Cultural Property (ICCROM); Auroville Earth Institute; The Grenoble National School of Architecture (ENSAG); etc. Their works through modern research have identified characteristics of earth, which are expressed in the advantages and disadvantages of earth construction.

13.6.1 Advantages of earth construction

A number of researchers who have done extensive work in the field of earth construction have identified, from their studies, unique advantages. It is reassuring to find several authors reaching the same conclusion. The findings of an exploration of the literature are summarised in Table 13.2.

Table 13.2 Advantages of earth construction

Merits	Publications
Low cost	Guillaud et al. (1985), Lal (1995), Easton (1998), Minke (2006), Morton (2007), Walker et al. (2005) and Zami and Lee (2007)
Encourages self-help with less skilled labour	Adam and Agib (2001), Minke (2006), Maini (2005) and Hadjri et al. (2007)
Good sound insulation	Morton (2007) and Hadjri et al. (2007)
Good heat insulation and fire resistance	Binici et al. (2007), Taylor et al. (2008), Hadjri et al. (2007), Adam and Agib (2001) and Walker et al. (2005)
Saves energy and no emission of CO_2	Minke (2006) and Glavind (2009)
Improves indoor air quality	Minke (2006), Hadjri et al. (2007), Lal (1995) and Walker et al. (2005)
Reusable	Minke (2006)
Low embodied energy	Keefe (2005) and Venkatarama Reddy and Jagadish (2003)
Capable of providing very strong and secured structure	Lal (1995), Rigassi (1985) and Walker et al. (2005)
Promotes culture, natural material	Frescura (1981)
Sufficiently available in most localities	Easton (1998), Adam and Agib (2001), Hadjri et al. (2007) and Lal (1995)

The embodied energy consumption of selected building materials was investigated by Keefe (2005). Results are shown in Table 13.3.

The embodied energy of earth is low at 5–10 kWh/m^3 compared to about 600 kWh/m^3 of concrete block.

13.6.2 Limitations of earth as building material

Earth has limitations too. Minke (2006) identified three disadvantages of earth in relation to other industrialised building materials, namely, non-standardised and non-water–resistant building material and shrinkage. The various limitations of earth identified by various publications are compiled in Table 13.4.

Table 13.3 **Embodied energy consumption of selected building materials**

Building material	Energy consumption (kWh/m^3)
Cement (PC)	2640
Fired brick (solid)	1140
Chipboard	1100
Lime	900
Plasterboard	900
Concrete block	600–800
Fired brick (perforated)	590
Calcium silicate brick	350
Natural sand/aggregate	45
Earth	5–10
Straw (baled)	4.5

Data from Keefe, L., 2005. Earth Building. Taylor & Francis Group, New York.

Table 13.4 **Limitations of earth construction**

Limitation	Publications
Non-standardised material	(Head (1980) and Minke (2006)
Non-resistant to water and less resilient as a construction material	Minke (2006), Lal (1995), Walker et al. (2005), Adam and Agib (2001), Blondet and Aguilar (2007), Hadjri et al. (2007) and Maini (2005)
Needs high maintenance	Hadjri et al. (2007)
Structurally limited	Hadjri et al. (2007), Maini (2005) and Adam and Agib (2001)
Suitable only for in situ construction	Walker et al. (2005)
Special skills required for plastering	Hadjri et al. (2007)

13.7 Surface protection of earth walls: Past and present

The protection of the surface of earth wall has always been of concern and has posed a challenge which has resulted in different solutions used in various cultures and environments. From simple ancient traditional solutions, surface protection of earth walls has developed into more sophisticated solutions with several options. There are two commonly used methods (no. 11 & 12) in modern earth construction practice, which were added to the 10 methods of surface protection identified by UNCHS (Habitat) (1986) and are presented in Table 13.5.

13.8 Need for further development of compressed earth block

Although efforts have been made to overcome the limitations of earth with improved results, work still has to be done. This is corroborated by the conclusion in Heathcote (1995) that durability of earth as a building material should be an area of further research in order for earth to gain wide acceptance in the contemporary context.

> *The durability of earth as a building material must be addressed if the material is to gain acceptance as a modern building material. It is undoubtedly the one area of earth wall construction which attracts the most concern from the general public. There is to date no conclusive evidence based on field performance to allay these concerns. (Heathcote, 1995)*

13.9 Durability assessment parameters

Morel et al. (2007) observed that though other parameters such as density and water absorption may be specified in design, compressive strength is a basic and universally accepted unit of measurement to specify the quality of masonry units and has become a universally accepted method as a result of the relative ease of undertaking this test in the laboratory. This opinion is lopsided, however, as it gives relevance only to the load-bearing capability of the earth wall and ignores the surface resistance of earth wall to surface erosion and water infiltration. The latter apparently is the main factor for the common hesitation in the use of earth as material of construction.

The surface protection has assumed a primary parameter for assessment of durability in the modern contest (Egenti et al., 2014). The outcome of efforts and developments in earth walls in terms of performance may be dependent on the parameters used for the assessment of durability and surface resistance of earth construction.

Table 13.5 **Analysis of old and new methods of surface protection**

S/no.	Name of method	Description of method	Merits	Demerits
1.	Weather boards	Coverings were attached to wall via a secondary structure in wood or metal. Covering may be of tiles, wood, cement-fibre, external insulating system, etc.	Low cost	Boards are easily blown away by wind No modern effect Provides breeding space and homes for insects
2.	Cladding	A cladding of prefabricated concrete elements is applied to the exposed surface of earth block wall. It was used in Germany in the 1920s	Impressive outer finishing	Sophisticated construction process High cost
3.	Facing	A facing of pebble or burnt brick is applied to the exposed surface of earth wall thereby creating a mixed wall. This earth wall protection process was common to the Mesopotamians, Asians and in the Pacific region	Outer surface of wall is protected	Strength of wall is not uniform and may result in unequal subsidence of wall or revetment of facing System not suitable for use in earthquake areas
4.	Integrated facing	A flat of 'L'-shaped facing elements of burnt clay are fitted on the rammed earth wall during construction or are included in adobe block during moulding. The dovetail fitting ensured adhesion of facing to the substrate	Outer surface of wall is protected from wear due to physical contacts	Water may penetrate the wall through the joints and cause damage to wall
5.	Twin layers	External surface of earth block is stabilised while still in the formwork/mould or partial stabilisation with layers of mortar or lime. Thickness of surface stabilisation is 2–3 cm. It was used in Burundi and Ouagadougou	Gives good result	Slow in production
6.	Inlay	Inlaid elements such as pebbles or flakes of stones, potsherds, brick flakes, shells, bottle tops, bottoms of bottles, etc., bound with mortar to form an outer wearing layer of earth wall	Protects outer surface	Production is demanding Requires a large supply of inlay element
7.	Surface treatment	External surfaces are treated by careful ramming with an extremely fine earth. This is intended to reduce the porosity of the earth. Tamping of surface with wooden paddle is common in Morocco and Yemen. The surface of wall can also be rubbed down with a stone	Protects outer surface	Continuous maintenance required for exposed walls

8.	Rendering	Earth wall may be rendered with earth, stabilised earth, or sand–cement or lime mortar. Other additives to rendering material may be bitumen, resin, etc.	Protects outer surface	Requires experience and good craftsmanship A nonporous rendering may retain interior moisture
9.	Painting	This is the painting of hydrated earth wall with cement, lime slurries or bitumen. The conventional paints, distempers and washes can also be used In traditional environment, a paste of cow or horse droppings/dung is used	Protects surface	Continuous maintenance is required
10.	Waterproofing coatings	Exposed surfaces are coated with water-repellent agents like silicon, resins in organic solution	Protects surface	Their efficiency is limited by cracks, and the risk of blistering It is expensive
11.	Impregnation	Earth is impregnated with products which confer certain qualities on the wall like impermeability, colour, etc. The products could be linseed oil, silicon, etc.	Protects surface	Continuous maintenance is required
12.	Stabilisation	Earth is stabilised by the addition of a binding agent to improve its water resistance capability. The commonly used binding agents include cement, lime and bitumen. Other stabilisers in traditional environments are natural stabilizers like the juice of the agave, opuntia cactuses, juice of boiled banana twigs, paste of rye flour, cow or horse droppings/dung, etc.	High percentage of cement, lime or bitumen considerably increases the water resistance of surface	Surface is degraded with time High percentage of stabilisation may be expensive
13.	Hyperpressure	The surface resistance and strength of masonry unit is increased by the application of extraordinary pressure by mechanical means (eg, hydraform, etc.)	Surface resistance is increased considerably when stabilised with small cement, bitumen or resins	Needs a sophisticated mechanical system, which may not be readily affordable in the context of low-cost and self-help construction system Surfaces wear with time if no high cement content

Adapted from UNCHS Habitat, 1986. Earth Construction Technology: Manual on Surface Protection. UNCHS (Habittat), Nairobi.

Appropriate durability assessment has been a subject of many research works (Heathcote, 1995; Walker, 1996; Venkararama and Jagadish, 1987). The wire brush test (ASTM, 1989) was developed for testing durability of earth-cement mixtures used in road construction. This test, first published in 1939, was later adapted for testing the durability of cement-stabilised earth in road construction. The test, which may be considered too severe, was used by Fitzmaurice (1958) as a guide to the durability of stabilised pressed earth bricks. He was of the opinion that the expectation of owners in relation to material performance is high and that the test provided appropriate assessment of durability. It involves essentially compacting an earth sample in a 100-mm-diameter mould to a depth of approximately 125 mm. Samples are cured for 7 days, oven dried and weighed. They are then placed in water for 5 h, then oven dried and brushed with a firm wire brush to remove all material loosened during the wetting and drying cycles. This process is repeated 12 times before final weight is recorded. The Portland Cement Association (1971) recommended weight loss less than 14% for well-graded sandy earth and 7% for clayey earth.

The spray erosion test was developed by the Commonwealth Scientific and Industrial Research Organisation (CSIRO). In this method, water is released under pressure 0.05 N/mm^2 (50 kpa) onto a defined area of block surface (Ciancio and Boulter, 2012). The pitting depth or eroded mass is measured as a percentage of the dry weight of the masonry unit or sample.

However, Ogunye and Boussabaine (2002) observed that most laboratory assessment methods for durability of earth wall are inconsistent with their field performance. He beliefs that a proper simulation of rainfall by drip test from a height of not less than 2 m may give a better result. The observation of Ogunye and Boussabaine (2002) suggest that the spray erosion test and the wire brush abrasion tests are too harsh for earth blocks. However, it is important to note that the reduction or lowering of the intensity of tests on earth wall could be responsible for the very common defect of earth walls.

The surface integrity test is another empirical test used by researchers to assess weather resistance properties. Pacheco-Torgal and Jalali (2012) exposed specimens of rammed wall to exterior weathering over a period of 20 years. He measured erodibility by the depth of pits formed over the years. A test for durability should be reliable and efficient for adoption in a contemporary context where very high performance is required for earth walls.

13.10 A sustainable option: Shelled compressed earth block

SCEB of Egenti et al. (2014) is discussed here as a case study of research intended to address challenges of sustainability of earth construction.

The *Shelled Compressed Earth Masonry Unit* is an earth block of two layers of different percentages of stabilisation. The inner layer/core was stabilised with zero or low cement; and the outer layer/shell was stabilised with higher cement content—all

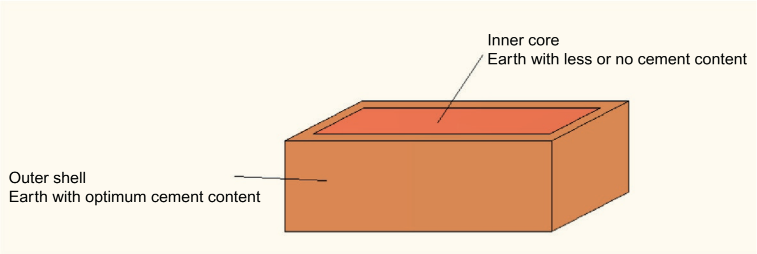


Fig. 13.1 Unit shelled compressed earth block (SCEB).

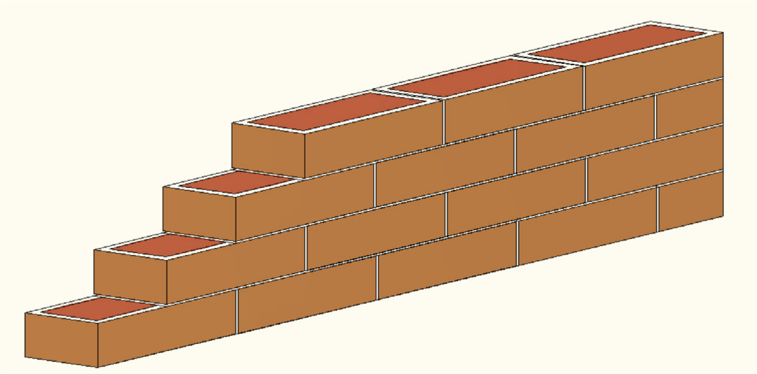


Fig. 13.2 SCEB masonry framework in running bond.

compressed mechanically into a single unit block. Fig. 13.1 shows an illustration of unit and block wall of SCEB. The concept evolved with the intention of giving adequate stabilisation to the exposed part of compressed earth block with less overall cement content and cost.

The arrangement of the blocks in stretcher or running bond, as shown in Fig. 13.2, shows a composition of masonry framework of shell in a supportive and interwoven structure and the inner core in a protective confinement (Egenti et al., 2014).

An industrial specimen of SCEB was produced using a simple mechanical kit and subjected to basic masonry tests.

13.10.1 Materials and methods

A cohesive, gravelly, lateritic soil of 38% passing a plasticity index of 15 was used for this investigation. The particle size distribution, plastic and liquid limit are as shown in Figs 13.3 and 13.4.

X-ray diffraction (XRD) analysis was performed to assess the mineralogical composition of Aviele laterite. The spectra of XRD powder pattern for the laterite soil is shown in Fig. 13.5. The soil is composed mainly of quartz, iron–magnesium–manganese (amphibole group) and kaolinite.

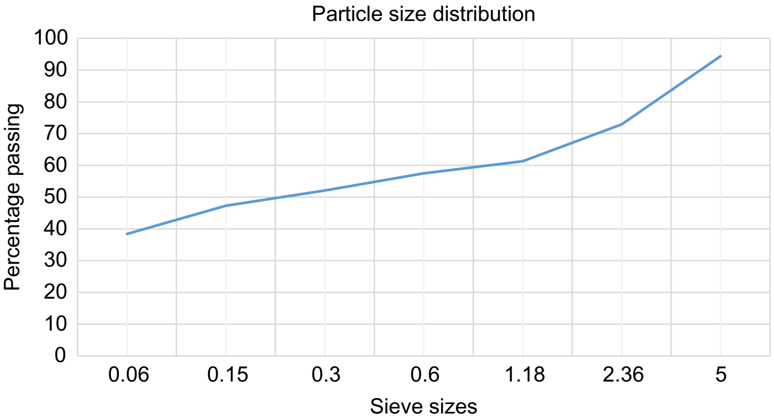
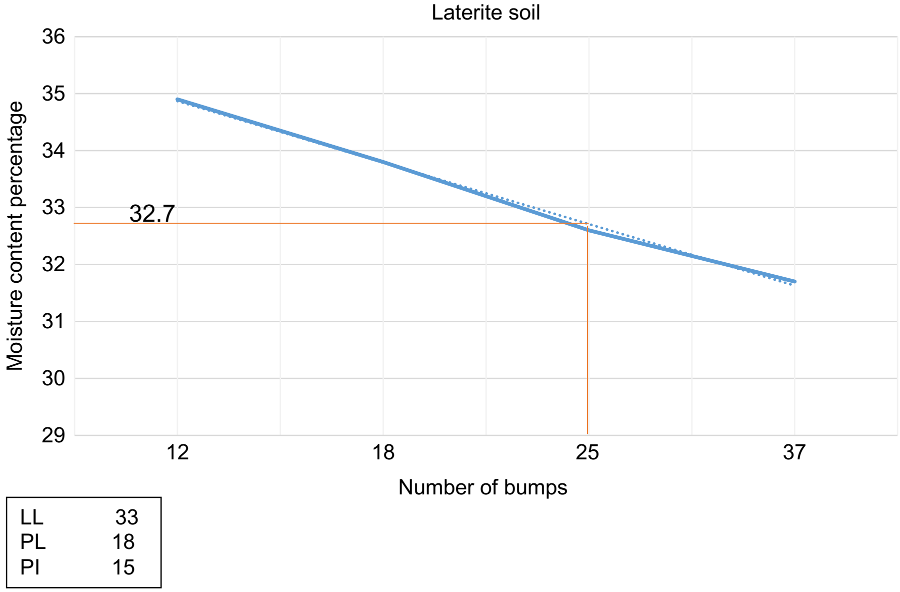


Fig. 13.3 Particle size distribution chart of laterite soil.



LL	33
PL	18
PI	15

Fig. 13.4 Plastic and liquid limit for laterite soil.

13.10.1.1 *Shelled compressed earth press*

The conventional compressed earth block presses produce blocks with one material mix. It was necessary that a mechanical kit (Fig. 13.6) be designed and fabricated to produce SCEB of two material mixes. A separator with two hoppers was introduced to the main mould to separate the shell from the core. The core material is fed from the central hopper, and the shell material is fed into the side hopper.

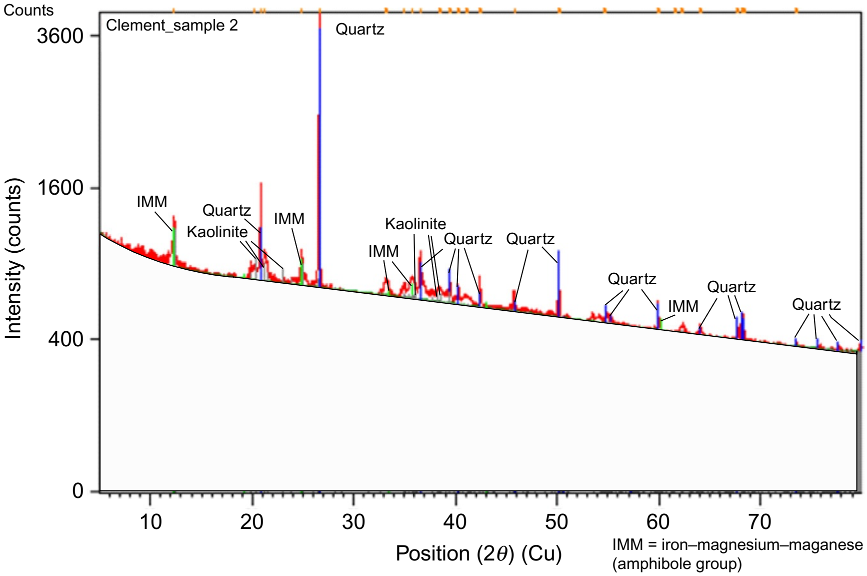


Fig. 13.5 XRD powder pattern for laterite soil.

Front view of mechanical kit

Back view of mechanical kit

Fig. 13.6 Different views of fabricated mechanical kit. 1—Mould cover; 2—mould; 3—gauge; 4—hydraulic pump; 5—lever; 6—hydraulic ram; 7—control valve; 8—vibrator; 9—hydraulic pressure pipes.

The design objectives were of a more robust mechanical setup capable of producing a 300-mm × 200-mm × 75-mm, hollow-shelled, compressed earth block. It was also capable of measuring compaction pressure of block produced using dial gauge as shown in Fig. 13.6.

13.10.1.2 The separator

A separator was designed and gauged to create a shell thickness of 20 mm in the interior and exterior facings of the block and 14-mm shell thickness on the headers. The separator of wide hoppers was constructed with 2-mm- thick plate as shown in Fig. 13.7. It separates the shell from the core.

13.10.1.3 The vibrator

The vibrator consolidates the content in the mould, particularly the shell with the narrow column of earth chamber. This alleviates the problem of space trap and honeycombing, which is detrimental to SCEBs. The vibrator was found to be very effective in this regard. The vibrator used was Wurges vibrator, 0.11 KW, UpM 2750. The pump stand, which incorporates the hydraulic gauge, was separated from the main frame to reduce the impact of the vibrator on the gauge.

13.10.1.4 Method of operation of mechanical kit

The method of operation is presented as follows:

With the control valve lowered, the hydraulic pump is applied to bring the bottom plate of the mould to the lower limit. The separator is then lowered into the mould and fed with the shell and the inner core materials. The vibrator is switched on for 5 s to consolidate the content. The space created by consolidation is topped up. The vibrator is switched on again while the separator is extracted from the moulds. The top of the content of mould is flattened, and the mould lid closed and clamped. With the control valve raised, the hydraulic pump is engaged to stress the content to the desired compaction pressure, which was readable on the gauge. The mould lid is opened and with the control valve raised, the block is ejected from the mould.

Fig. 13.7 The separator of shell from the core.

Fig. 13.8 SCEB produced with mechanical kit.

Typical specimens of hollow SCEB produced with the mechanical kit are shown in Fig. 13.8. The two holes reduced the overall weight of block by 24%. It is also anticipated that the hollowed nature of the block will accommodate any expansion of the inner core material.

13.10.1.5 Properties of compressed earth blocks

The current investigation is based on standard tests stipulated in Eurocode 6 and American Society for Testing and Materials (ASTM) as appropriate for masonry units and of direct relevance to strength, durability/surface resistance, water absorption and structural stability. The tests were conducted within a range of compaction pressure (1.7–3.5 N/mm^2) and methods adopted in conducting these tests are reported in this section.

The physical properties of the new masonry unit were investigated, commencing with a test of *Net Dry Density* with varied compaction pressure. The capability of the mechanical kit to measure the degree of compaction at the point of production of the block provided the opportunity of varying compaction pressure and measuring the effect on density. The outcome of the investigation and analysis was an understanding of the effect of compaction pressure on durability of masonry unit. This test was conducted in accordance with BSI (2000). Specimens of SCEBs were produced with a compaction pressure varied from 1.7 to 3.5 N/mm^2. This range was selected because it falls within a low-cost and self-help construction method and can be delivered with a simple compressed earth block press. Higher pressure requires sophisticated mechanical equipment, with the accompanying high cost of purchase and operation. SCBs of Nigerian common mix ratio of 1:4:2 were also produced for purpose of comparison.

A *compressive strength* test was conducted. Sets of six specimens of SCEB were produced from the Aviele laterite soil. Ten percent cement content was added to the laterite soil for the shell, while 3% cement was added to the laterite soil for the core. For the purpose of comparison, sets of SCBs specimens were produced with mechanical kit for mixes 1:4:2 (cement/fine sand/coarse sand), vibrated as usual for sandcrete blocks. A compressive strength test was conducted as described in BSI (2011a). The compressive strength readings were normalised by 0.75 in consideration for shape factor as specified in appendix A of BSI (2011a).

The *initial rate of water absorption* (IRWA) test was conducted to assess the water absorption behaviour of SCEB blocks. The rate of absorption determines how soon, if at all, water from the exterior infiltrates or causes dampness of the inner wall surface. Sets of six specimens of SCEB were produced from the Aviele laterite soil with compaction pressure varied from 1.7 to 3.5 N/mm^2. Ten percent and 3% cement content was added to Aviele laterite soil for the shell and core, respectively. For the purpose of comparison, a set of six specimens of SCBs of the common mix (1:4:2) in Nigeria was also produced for this test. The IRWA was conducted and calculated in accordance with BSI (2011b). The study was taken further into an assessment of rate of water absorption over a period of 10 min.

A test of *stress and strain* was carried out to measure the strain of SCEB under stress. It is particularly relevant in a masonry unit of two layers of different material strength to observe how the two layers behave or co-work in resisting the applied load. Specimens of SCEB of shell composition of 10% cement content and inner core of 3% cement content were produced from Aviele laterite soil with different compaction pressure from 2 to 3.5 N/mm^2. The specimens were conditioned and subjected to varying stresses while measuring corresponding displacements.

The specimens of SCEBs were tested for *flexural strength* to assess the capability of the new block to withstand the wind load of the tropical environment. Jayasinghe and Mallawaarachchi (2009) was of the opinion that since high lateral loads could be caused under unusual conditions such as cyclones, floods or earthquake, strength data on flexural strength was important to allow adequate provision for such extreme forces of nature. Several blocks were produced for the purpose of erecting wall panels for the flexural strength test in accordance with BSI (1999). A flexure testing kit was set up as shown in Fig. 13.9 and illustrated in Fig. 13.10.

The flexural strength was calculated from the results of the characteristic panel compressive strength of specimens with the following formula in BSI (1999):

$$f_{xi} = \frac{3F_{i,\max}\left(l_1 - l_2\right)}{2bt_{\mathrm{u}}2}\,\mathrm{N/mm^2}$$

where f_{xi} is the flexural strength of masonry panel (N/mm^2), $F_{i,\max}$ is maximum force at failure (maximum flexure load) (N), l_1 is the spacing of the outer bearings (mm), l_2 is the spacing of the inner bearings (mm), b is height of masonry panel (mm) and t_{u} is the width of masonry unit (mm).

For the flexure machine used, $l_1 = 350$ mm and $l_2 = 650$ mm (Fig. 13.5).

For the masonry panels used $b = 800$ mm and $t_{\mathrm{u}} = 200$ mm (Fig. 13.5).

Fig. 13.9 Flexural strength apparatus. 1—Outer bearing; 2—inner bearing; 3—hydraulic press; 4—radial load gauge; 5—hydraulic hand pump; 6—SCEB wall panel or wallette.

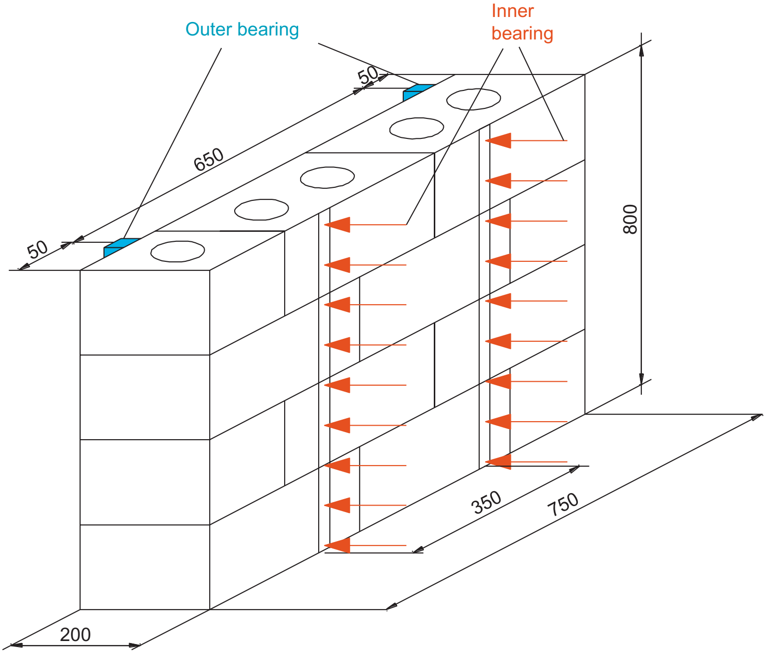


Fig. 13.10 Inner and outer bearings position for flexural tests of masonry panel.

The preceding tests have measured the load-bearing characteristics which have been the main focus of most research in earth construction. The *durability/surface resistance tests* are particularly crucial to social acceptability, which is the focus of this research. The test was conducted in four different types of tests: namely, the drip test, hand test, wire brush test and block integrity test.

13.10.2 Results and discussions

13.10.2.1 Net dry density

The change in compressive pressure on the average net dry density of blocks produced with different compaction pressure was calculated with details shown in Table 13.6 and charted for visual appreciation in Fig. 13.11.

Table 13.6 Effects of compaction pressure on net dry density

Compaction pressure (N/mm^2)	Unit mass, $m_{dry,u}$ (g)	Net volume of unit, $V_{n,u}$ (mm)^3	Net dry density, $\rho_{n,u}$ (kg/m^3)	Average net dry density, $\rho_{n,u}$ (kg/m^3)
1.7	10,395	504.2×10^4	2063	2017
	10,085	501.9×10^4	2009	
	10,105	510.7×10^4	1979	
2	10,600	520.2×10^4	2038	2020
	9980	499.7×10^4	1997	
	10,400	507.1×10^4	2051	
2.5	10,595	523.9×10^4	2022	2029
	10,110	502×10^4	2014	
	10,305	509.3×10^4	2023	
3	10,655	517.9×10^4	2057	2033
	10,145	501×10^4	2025	
	9975	494.6×10^4	2016	
3.5	10,300	489.2×10^4	2105	2075
	10,155	491.9×10^4	2064	
	10,390	505.3×10^4	2056	
Sandcrete block 1:4:8	7395	501.3×10^4	1676	1508
	7023	484.3×10^4	1450	
	6937	496.2×10^4	1398	

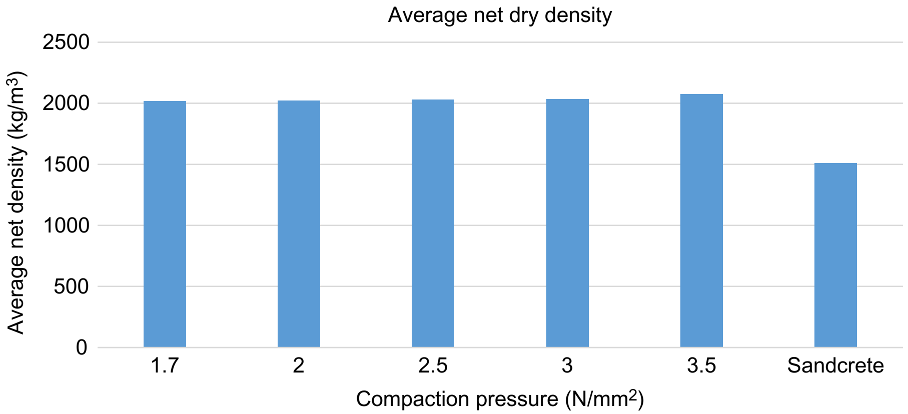


Fig. 13.11 Changes in average net dry density with increase in compaction pressure.

Dry density slightly increases with increase in the compaction pressure with which SCEB was produced. The National Building Code of Nigeria specified 3 N/mm^2 as minimum compaction pressure for compressed earth block of 5% cement content (MLHUD, 2006). A comparison of net dry density with SCB revealed a lower average density of 1508 kg/m^3 of SCB. While compaction pressure apparently does not have a significant effect on the rate of loss of moisture from SCEB, it does have a very significant effect on the net dry density of the block. The net dry density increases with increase in the compaction pressure. The net dry density of SCB is significantly lower. While this may be an advantage in terms of dead weight, an assessment of compressive strength is necessary.

13.10.2.2 Compressive strength

The results of the compressive strength at different stages of strength development of the specimens are shown in Table 13.7 and Fig. 13.12. Strength development can be seen to progress with days of curing. A compressive strength of 2 N/mm^2 at 7 days implies that block can be used for wall construction at this stage.

Failure pattern: The failure pattern of the industrial model SCEB is characterised by shearing of shell as shown in Fig. 13.13.

The changes in compressive strength with varying compression pressure and the comparison with SCBs are shown in Fig. 13.14.

The compressive strength increases with an increase in compaction pressure. There is an appreciable rise in compressive strength with 3.5 N/mm^2 compaction pressure. The common mix of SCB (1:4:8) has a very low compressive strength of 1.58 N/mm^2 while the plaster mix 1:4:2 is only comparable with the 2.5 N/mm^2 compaction pressure of SCEB.

Compaction pressure has a significant impact on compressive strength of the SCEB. The compressive strength increases with an increase in the compaction pressure. The common SCB (mix 1:4:8) in Nigeria exhibits a very low compressive strength. The lowest compressive strength of SCEB sample produced with 1.7 N/mm^2

Table 13.7 Strength development of SCEB with changes in compaction pressure and materials

Compaction pressure (N/mm^2)	Normalised compressive strength, 7 days (N/mm^2)	Normalised compressive strength, 14 days (N/mm^2)	Normalised compressive strength, 28 days (N/mm^2)
1.7	2.42	2.9	2.93
2	2.48	3.06	3.19
2.5	2.8	3.12	3.32
3	2.93	3.32	3.57
3.5	3.19	4.08	4.46
Sand–cement blocks (1:4:8)	1.2	1.41	1.58
Sand–cement blocks (1:4:2)	2.93	3.05	3.23

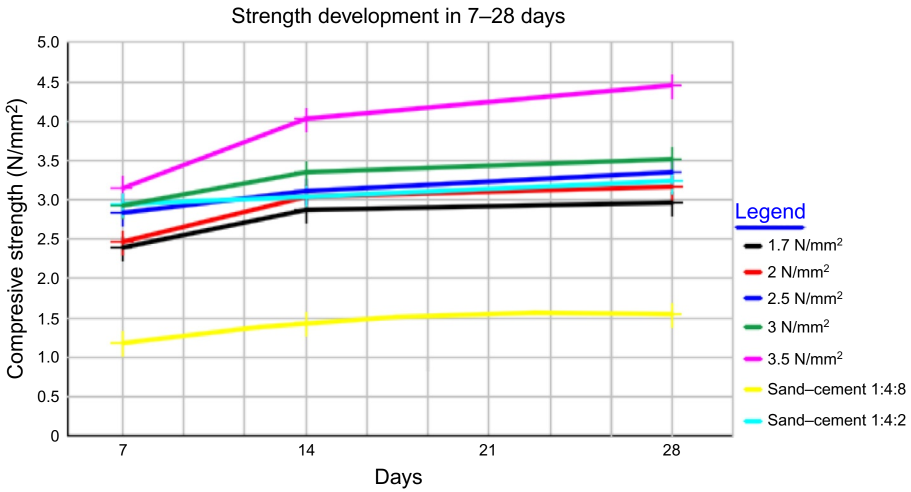


Fig. 13.12 Strength development in 7–28 days.

Fig. 13.13 Failure pattern of SCEB.

compaction pressure, which is 2.93 N/mm², is even adequate for simple buildings. NZS 4298:1998 specified 1.3 N/mm² minimum normalised compressive strength (Standards New Zealand, 1998). This suggests a wide range of compaction pressure which may be tolerable for SCEB. It is, however, subject to the results of the tests for rate of water absorption and water erosion.

13.10.2.3 Initial rate of water absorption

The result of the IRWA of SCEB produced with different compaction pressures (Fig. 13.15), and in comparison with SCB, is shown in Figs 13.16 and 13.17. The higher capillary rise of moisture at the upper corner of blocks indicates less compaction of top of block, hence more absorption.

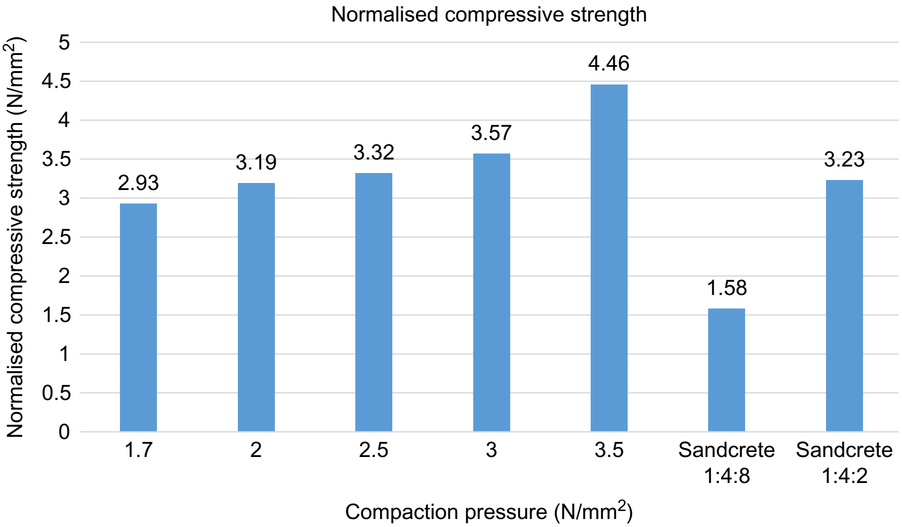


Fig. 13.14 Changes of compressive strength with increase in compaction pressure and comparison with sand–cement blocks.

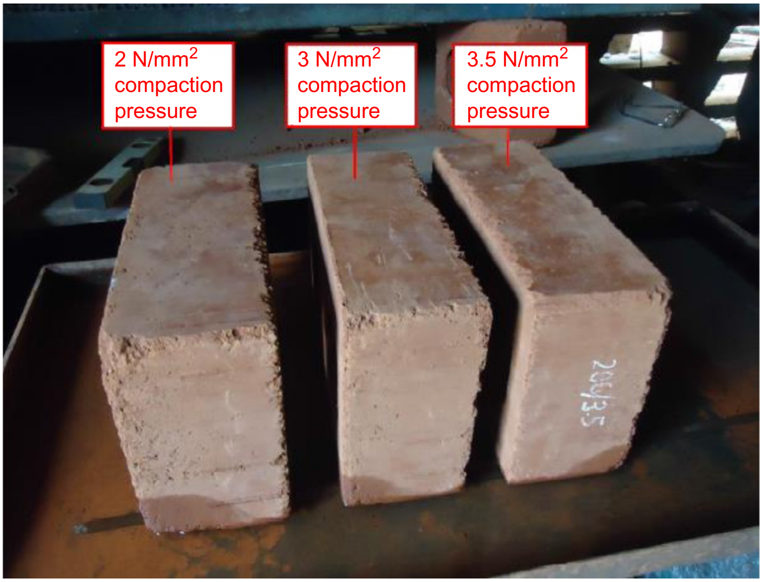


Fig. 13.15 Test of initial rate of capillary absorption of SCEB.

The IRWA reduces significantly with increase in compaction pressure of SCEB. The SCB has a high rate of water absorption. This also shows that though the compressive strength of 1.7 N/mm^2 compaction pressure was adequate for simple buildings, it may be advisable to use a compaction pressure of 2.5 N/mm^2 and above for better performance when subjected to water. The study of the rate of water absorption over a period of 10 min was also made, and the result is shown in Fig. 13.16.

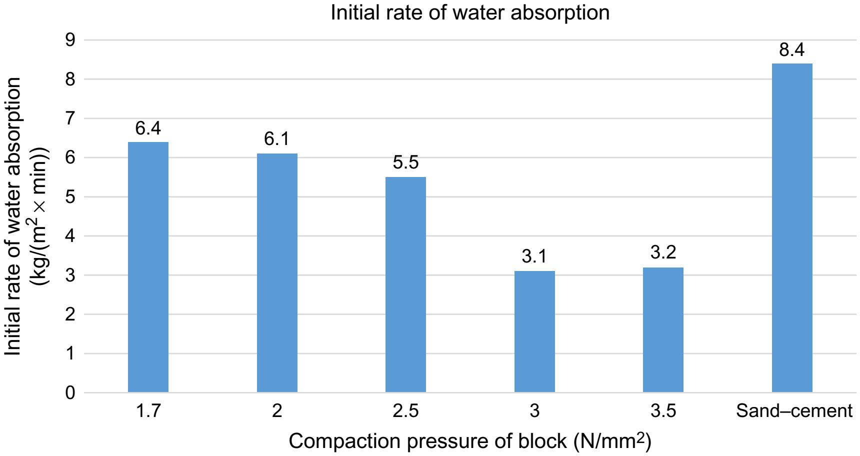


Fig. 13.16 Initial rate of water absorption of SCEB and sand–cement block.

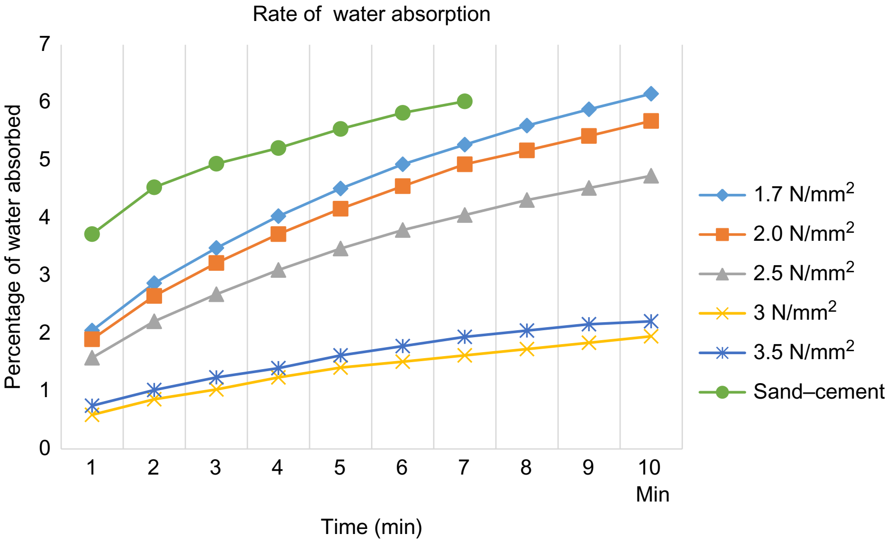


Fig. 13.17 Rate of water absorption of SCEB and sand–cement for 10 min.

The sand–cement absorbed consistently to almost saturation in 7 min. The rate of water absorption of SCEB reduces with increase in compaction pressure. The rate of water absorption of SCEB is reduced when the compaction pressure is increased with a minimal absorption rate for 3 and 3.5 N/mm^2. This shows that though the compressive strength of 1.7 N/mm^2 compaction pressure was adequate for simple buildings, it may be advisable to use a compaction pressure of 2.5 N/mm^2 and above for better performance with water. A comparison with SCBs (as observed in the charts) revealed that the rate of water absorption of SCB is significantly higher than in the SCEB of the earlier specified compositions.

13.10.2.4 Stress and strain

The measured displacements of specimens produced with different compressive pressure at different stress application stages are shown in Table 13.8 and Fig. 13.18.

Table 13.9 and Fig. 13.19 show strain at different stress applications on specimens produced with different compressive pressure.

The graph shows the compressive strength at deformation increased with increase in compaction pressure. This is expected, and one can conclude that there was no unusual behaviour exhibited by the industrial model when loaded to deformation.

Table 13.8 **Measured displacements with different stress applications**

Specimens	**Displacements with progressive stress application of 0–5.5 N/mm², δ (mm)**											
	0	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.3
2 N/mm² CP	0	0.5	1	1.3	1.5	1.7	2	2.5	3.5			
2.5 N/mm² CP	0	0	0.4	0.6	1	1.4	1.5	1.7	2.2	4		
3 N/mm² CP	0	0.3	0.5	0.8	0.8	1	1.2	1.3	1.8	3.8		
3.5 N/mm² CP	0	0.8	1.3	1.4	1.8	1.9	2.7	2.8	3	3.3	3.4	5.4
Sand–cement	0	0.9	1.2	1.3	1.5	1.7	1.9	2.2	2.5			

Note: CP—compaction pressure.

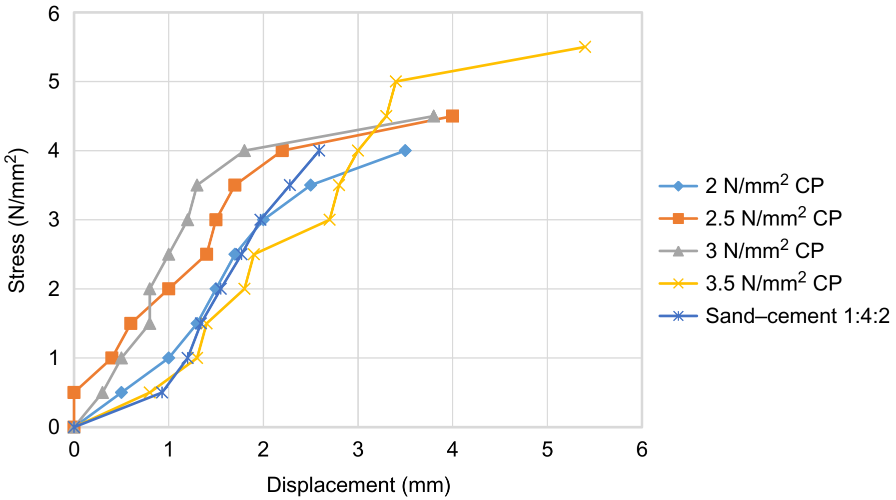


Fig. 13.18 Displacement at different stress applications.

Table 13.9 Strain at different stress applications

Specimens	Original height of specimen (mm)	Strain with progressive stress application of 0–5.5 N/mm², ϵ_e (%)											
		0	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5
2 N/mm^2 CP	109.5	0	0.46	0.91	1.19	1.37	1.55	1.83	2.28	3.2			
2.5 N/mm^2 CP	110	0	0	0.36	0.55	0.9	1.27	1.36	1.55	2	3.64		
3 N/mm^2 CP	109.8	0	0.27	0.46	0.73	0.73	0.91	1.09	1.18	1.64	3.46		
3.5 N/mm^2 CP	107.8	0	0.74	0.93	1.3	1.67	1.76	2.5	2.6	2.78	3.06	3.15	5.01
Sand–cement	96.5	0	0.9	1.2	1.34	1.55	1.77	1.97	2.28	2.59			

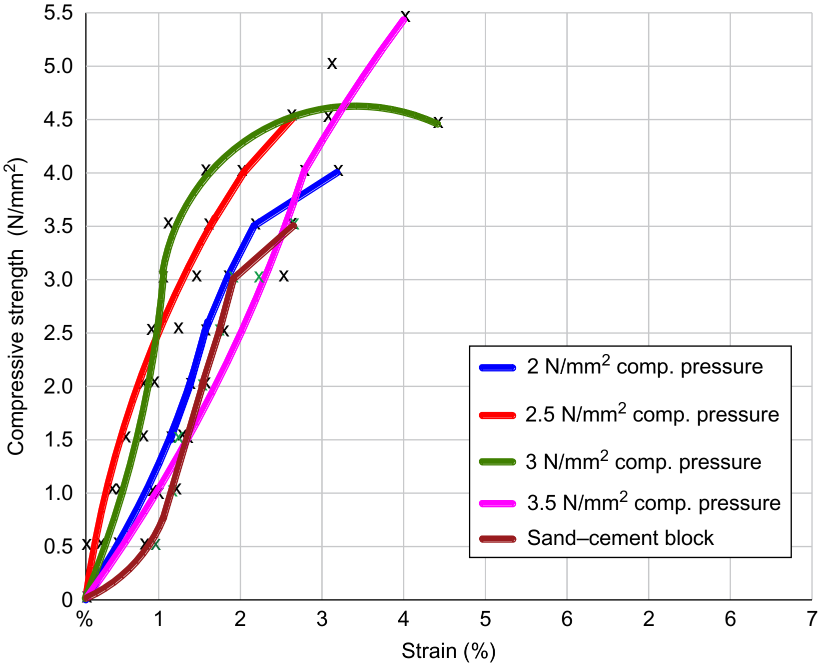


Fig. 13.19 Strain curves for shelled compressed earth blocks of different compressive pressure.

13.10.2.5 Flexural strength

The result for the five valid tests for SCEB units and panel specimens are shown in Table 13.10.

The mean flexural strength was characterised by using the formula below as stated by the BSI (1999).

$$f_{xk} = f_{\text{mean}} / 1.5$$

where f_{xk} is characteristic flexural strength.

This was applied to the f_{mean} of 1.47 N/mm² in the preceding result and got the f_{xk} as 0.98 N/mm² in Table 13.10. The result of the five valid tests for *SCB* units and panel specimen are shown on Table 13.11.

The average unit normalised compressive strength and characteristic flexural strength of SCEB masonry panel specimen was found out to be 4.7 N/mm² and 0.98 N/mm², respectively. In comparison, the respective values of unit normalised compressive strength and characterisitic flexural strength of the sand–cement masonry specimens were measured to be 3.42 and 0.71 N/mm², respectively. These set SCEB higher on a scale of capability (Fig. 13.20) to withstand unusual weather conditions like wind storms, floods and earthquakes.

A typical failure pattern for this test is shown in Figs 13.21 and 13.22.

Table 13.10 Results of tests of SCEB for characteristic flexural strength

Specimen	Normalised compressive strength of unit SCEB (N/mm^2)	Compressive strength of masonry panel, $F_{i,\max}$ (N)	Flexural strength of masonry panel, f_{xi} (N/mm^2)	Mean flexural strength, f_{mean} (N/mm^2)	Characteristic flexural strength, f_{xk} (N/mm^2) ($f_{xk} = f_{\text{mean}}/1.5$)
Panel R1	4.81	60,520	1.43	1.47	0.98
Panel R2	4.42	59,400	1.41		
Panel R3	4.68	66,150	1.57		
Panel R4	4.78	63,860	1.51		
Panel R5	4.63	61,220	1.44		

Table 13.11 Results of test of sand–cement block for characteristic flexural strength

Specimen	Normalised compressive strength of unit sand–cement block (N/mm^2)	Compressive strength of masonry panel, $F_{i,\max}$ (N)	Flexural strength of masonry panel, f_{xi} (N/mm^2)	Mean flexural strength, f_{mean} (N/mm^2)	Characteristic flexural strength, f_{xk} (N/mm^2) ($f_{xk} = f_{\text{mean}}/1.5$)
Panel SC1	3.11	43,998	1.04	1.07	0.71
Panel SC2	3.61	43,184	1.02		
Panel SC3	3.58	48,091	1.14		
Panel SC4	3.0	46,426	1.10		
Panel SC5	3.78	44,507	1.05		

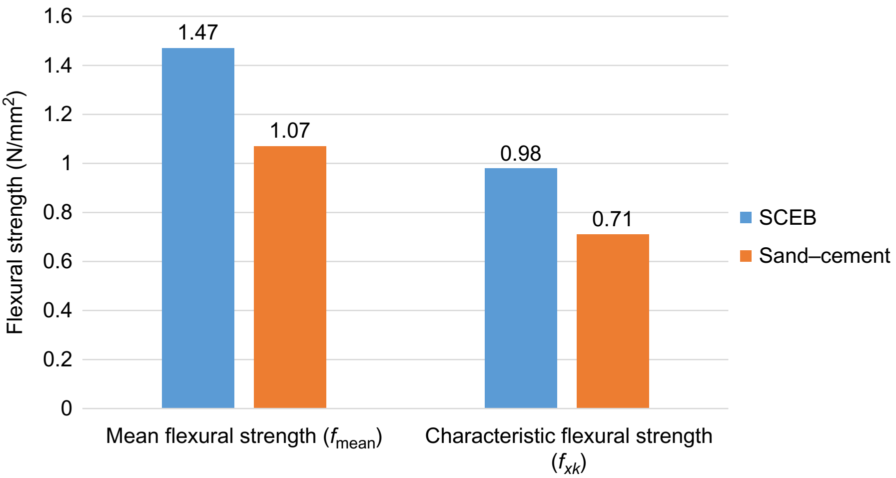


Fig. 13.20 Comparison of characteristic flexural strength of SCEB (shelled compressed earth block) and sand–cement block.

Fig. 13.21 Typical failure pattern of flexural strength test.

Fig. 13.22 Typical failure pattern (rear view).

The research work of Jayasinghe and Mallawaarachchi (2009) indicated characteristic flexural strength of different types of walling materials like burnt clay bricks, cement stabilised earth solid and hollow of different bonds, and rammed earth in a range of about 0.3–2 N/mm^2.

13.10.2.6 Surface resistance

The *wire brush/abrasion test* revealed the limitations of stabilised compressed earth blocks produced with less compaction pressure. The percentage of loss was higher with lower compaction pressure as shown in Fig. 13.23.

The abrasion property of SCEB improves with increase in compaction pressure. The surface resistance of 3 and 3.5 N/mm^2 of SCEB compaction pressure is higher than SCB of 1:4:2. An all-clear durability is hence achievable with 3–3.5 N/mm^2 compaction pressure. The preceding test results and performance of the proposed model of masonry unit in Nigeria in comparison with the Nigerian commonly used SCBs are summarised in Table 13.12. The results are relative to Aviele laterite soil of 38% clay and silt content.

13.11 Future trends

The majority of current researches are focussed on the adaptation of earth construction to local environments by the use of waste material in such local environments as additive to earth for building purposes. Villamizar et al. (2012), in Colombia, investigated the addition of coal ash and cassava peels on the engineering properties of compressed earth blocks. Lima et al. (2012) were inspired to utilise the sugarcane bagasse ash, an industrial waste in Brazil, as an additive to compressed earth blocks.

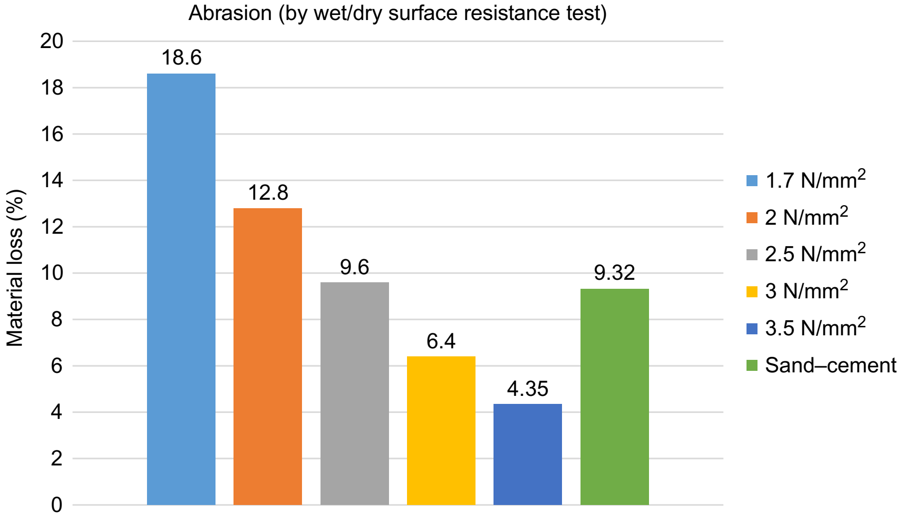


Fig. 13.23 Graphical presentation of material loss by wet/dry surface resistance test.

Table 13.12 **Summary of test performance of various types of blocks**

Block type	Material composition (% cement)		Compressive strength (N/mm²)	Initial rate of water absorption (kg/(m²×min))	Flexural strength of masonry (N/mm²)	Durability/surface resistance	
	Shell (%)	Core (%)				Abrasion (%)	Drip (%)
SCEB	10	3	4.46	3.2	1.07	4.35	0
Vibrated sand–cement 1:4:2			3.23	5.3	0.71	9.32	0

Future research in earth construction may develop in recognition of the challenges of true sustainability of earth construction techniques, and thus focus on concepts that will balance the three factors of sustainability: environmental, social-cultural and economic. The trend is towards creative adaptations and innovative concepts that could optimise the potential of earth construction. Heath et al. (2012) suggested 'appropriate detailing' as a requirement for modern earth construction. Egenti et al. (2014) conceptualised the shelled earth block as discussed in the foregoing.

The sustainability of compressed earth wall was critically analysed in the context of contemporary needs and performance. The investigation revealed that though the environmental and economic factors were addressed by traditional earth construction techniques, the social factor suffers from unacceptability in urban areas. Attempts by most modern developers to address the social factor have resulted in the use of high cement content, thereby threatening the environment and economic angles traditionally associated with earth construction. A recent innovative research effort was taken as a case study as typical of complete sustainability found in shelled compressed earth construction technology. Results of tests conducted on SCEB revealed a very significant change in compressive strength from 1.9 to 5 N/mm^2 by the introduction of a shell of 3% cement content and about 6.5 N/mm^2 with 5% cement in shell. The IRWA by capillary action reduced from 1.9 $kg/(m^2 \times min)$ for 0% cement content in shell, to 0.8 $kg/(m^2 \times min)$ with 10% cement in shell. The percentage loss by abrasion reduced from 69% for 0% stabilisation to 4.5% with 10% cement stabilisation of shell. Thus, a high performance required by contemporary buildings in urban areas is achievable with a reduction in overall cement content and cost.

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Sustainability of bituminous materials

14

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14.1 Introduction to bituminous materials

Bituminous materials, also known as asphalt mixtures, are amongst the oldest engineering materials. Their use as binding and waterproofing materials can be dated back to 3000 BC, and later the Romans developed roads in many parts of their empire, marking the first global use of bituminous binders in road paving technology. Eurobitume (2015) reported that over 90% of all the roads in Europe are surfaced with bituminous materials.

Bituminous materials (asphalts) typically comprise three major constituents: aggregates (80–95%), filler (2–8%) and bituminous binders (3–7%); additives are sometimes used to enhance the properties of the mixture. Aggregates and fillers are the predominant constituents of bituminous mixtures and provide a skeleton which contributes to the stability to the mixture.

Bitumen plays an essential role in the construction and strengthening of bituminous pavements where it is predominantly used as the binder. In North America bitumen is commonly known as 'asphalt cement' or 'asphalt'; this should not be confused with the term 'asphalt' as just described which has been used widely outside North America. Bituminous binder should be of sufficient quantity to coat the aggregates and filler; to provide good workability during mixing, laying and compaction of the mixture; and to promote flexibility of the mixture in traffic-loading conditions and in terms of durability during its expected lifetime.

14.2 Bituminous binders

Bituminous binders can be found in three main forms: as naturally occurring materials, as refined products and as processed ('synthetic') materials. Amongst these forms, refined products are mostly used in paving industries. Natural asphalts are too hard to be used immediately and often vary greatly in their composition, therefore requiring further processing or refining prior to being used in bituminous mixtures. Consequently, natural asphalts have limited use in paving applications. Both naturally occurring asphalts and refined bitumen products are obtained from limited resources; therefore they are not renewable and must be used wisely. More recently, however, a number of trials have used binders from renewable resources for paving applications, although these binders aren't seen as being commercially viable.

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14.2.1 Natural asphalt

Natural asphalt typically is found in the form of rock impregnated with 5–15% bitumen. It was discovered in France and Switzerland in the 18th century and was found to provide a durable and waterproof layer after mining, grinding and heating to form a paste or epuré. In the 19th century it was discovered that a naturally occurring asphalt located in a ‘lake’ in Trinidad, West Indies (Trinidad Lake Asphalt—TLA) could be added to (and eventually replace) the rock asphalt, while enhancing the durability of the mixture. TLA consists of a mixture of bitumen and minerals of the following composition: soluble bitumen (53–55%), mineral matter (36–37%) and others (9–10%). Other rock asphalts can be found in Indonesia (Buton asphalt) and Midwestern United States (Gilsonite or Uintate).

14.2.2 Refined bitumen

Most bitumen is manufactured from crude oil. It is generally accepted that crude oil originates from the remains of marine organisms and vegetable matter (principally algae) deposited underwater with silt (inorganic matter) anywhere from one million to one billion years ago. Over millions of years, organic and inorganic material accumulated into layers hundreds of metres thick, the weight of the upper layers compressing the lower layers into sedimentary rock. The organic–inorganic mixture first converted into what is commonly known as kerogen, a consolidated organic—mineral matrix with little or no solubility in organic solvents. It is believed that the conversion of kerogen into the hydrocarbons of crude oil resulted from the application of heat from within the earth's crust and the pressure applied by the upper layers of sediments. These physical and chemical changes (diagenesis) produced different materials of different properties because the temperatures and pressures vary at different depths, and over different time periods.

As further layers of sediment were deposited on the sedimentary rock where the oil formed, the additional pressure squeezed the oil into porous rock deposits overlain by impermeable rock, forming oil and gas reservoirs. Also, different migration conditions (from formation to reservoir) may have resulted in different properties in the resulting crude oil. Some higher plant forms were also buried and became, through the process of diagenesis, petroleum. Here the plant cuticle (water-resistant layer on leaves) is a waxy substance that often survives to become part of the crude oil. Hence high wax crudes are another common variation in the composition of crude oil. Refer to Tissot and Welte (1978) for a more detailed discussion about petroleum formation. Fracture of the earth's crust may allow oxygen (air), water and bacteria to invade the petroleum formation. Bacteria particularly have a selective appetite for certain crude oil fractions and may further alter composition.

The four main oil-producing areas in the world are the Middle East, the Americas, the countries of the former Soviet Union and the Caribbean. As illustrated by the preceding basic view of petroleum formation, crude oils can vary widely in both physical and chemical properties. Furthermore, crude oil refining first separates out the high value products—petrochemicals, gasoline, diesel, jet fuel and lubrication oil—each of which have similar properties regardless of the crude oil or the refinery process. In a sense, therefore, the leftovers, the residua or bitumens, are concentrated versions of the differences among crude oils. Nearly 1500 different crude oils are produced worldwide. Only a few of these are considered suitable for the manufacture of bitumen. In the United Kingdom, bitumens are principally manufactured using either Middle Eastern or South American crudes (Morgan and Mulder, 1995).

The preceding discussion illustrates that bitumens from different crude oil stocks should be expected to be substantially different from each other. The mere fact that two or more bitumens are processed in refineries to the same (say) viscosity does not make them interchangeable with one another in all circumstances. Their chemical compositions do vary from one source to another; hence their reactions and interactions with their surroundings (service environment) will also vary.

Furthermore, bitumen should not be confused with tar. Although bitumen and coal tar may have a similar appearance and adhesivity, they are fundamentally different in origin, chemical composition and properties.

Coal tar is a by-product of gas and coke production which is produced by heating coal to extremely high temperatures. It was widely used as binder in asphalt mixtures for roads at the beginning of the 20th century but has since been replaced by refined bitumen.

14.2.3 Processed binders from renewable sources

As described previously, bitumen is mostly produced from skilful engineering of oil processing. Because oil is a finite resource, any issue associated with the existence of oil can be extended to the supply of bitumen; that is, a high price for crude oil will also mean a high price for bituminous products. However, recent developments suggest that it is possible to produce binder from renewable sources. A number of organisations have developed this binder type; two of them are presented here as examples.

Asphalt made with vegetable-based binders were patented by Colas SA in France in 2004 under the trade name Végécol (Ballié et al., 2004). This product was claimed to be 100% vegetable binder as no petroleum polymer was used in the formulation. This product was developed to achieve levels of performance comparable to those of asphalt while being entirely composed of renewable raw materials from agricultural resources. In tests Végécol exhibited improved mechanical properties compared to control asphalt samples which were coated with conventional bitumen. Végécol can also, through its translucency, be produced in the natural colours of aggregates or in colours of any shade. By using renewable raw materials, Végécol contributes to the goals of sustainable development. The material is claimed to have good cohesive and elastic properties, high resistance to rutting and fatigue, good resistance to hydrocarbons/fuel and lower mixing temperatures as compared to conventional bitumen.

Shell Oil Company has test trialled two public roads using BioFlux, a vegetable oil-based binder, for use in ‘hot mix asphalt’ (HMA) and ‘hot surface dressing’ applications in Norway (Andersen et al., 2008). The trial concluded that the performance of materials manufactured using biofuel binders was at least similar to that of conventional materials. Other benefits included a more pleasant odour than the stronger odours emitted by conventional HMA.

14.3 Characteristics of bitumen

Bitumen is a viscoelastic material, which means that bitumen properties (such as stiffness, viscosity or consistency) change with any change in temperature and/or loading condition. Consequently, understanding the viscoelastic behaviour of bituminous binders is considered to be essential in the design of bituminous mixtures. Furthermore, bitumen is a 'live' material. The properties keep on changing from the time of production and on throughout the material's service life, which involves a combination of complex chemical and physical processes, including 'evaporation' of more volatile, lighter and oily components which collectively lead to hardening of the binder.

Bitumen hardens considerably during asphalt production, transportation and paving operations, and subsequent age hardening occurs at ambient temperatures over many years. Moderate age hardening after paving increases the stiffness and actually improves the load-spreading ability of the pavement. However, excessive hardening reduces the flexibility of the material and may lead to embrittlement. Embrittlement of bitumen significantly increases the vulnerability of the bituminous mixture in the pavement to fatigue cracking, thermal and reflective cracking and a reduced capacity for healing.

Bitumen properties are generally classified by and tested for their consistency at intermediate and elevated service temperatures, for their brittleness at low service temperatures, for their consistency of temperature dependence, for their durability of consistency at intermediate and elevated service temperatures and for their content of dangerous regulated substances. Standardised methods to characterise paving grade bitumen can be found in EN 12591 (CEN, 2009a); some key characteristics are summarised in Table 14.1. Paving grade bitumen is neat or unmodified bitumen which is mostly used in the production of asphalt paving materials. The consistency of the temperature dependence of paving grade bitumens is commonly represented by a calculated penetration index (I_p) based on Annex A of EN 12591, which considers the change in the penetration values between a temperature of 25°C and that of the ring and ball softening point, as illustrated in Eq. (14.1).

Table 14.1 Characteristics of paving grade bitumen in accordance with EN 12591

Characteristics	Test method	Description
Consistency at intermediate service temperature	EN 1426 (CEN, 2015a)	Penetration
Consistency at elevated service temperature	EN 1426 (CEN, 2015a) EN 1427 (CEN, 2015b) EN 12596 (CEN, 2015c)	Penetration Softening point Dynamic viscosity
Brittleness at low service temperature	EN 12593 (CEN, 2014a)	Fraass breaking point (FBP)
Durability of consistency at intermediate and elevated service temperatures	EN 12607 (CEN, 2014b,c)	Rolling Thin Film Oven Test (RTFOT), Thin Film Oven Test (TFOT)

$$I_p = \frac{20 \times t_{R\&B} + 500 \times \lg P - 1952}{t_{R\&B} - 50 \times \lg P + 120} \qquad (14.1)$$

where $t_{R\&B}$ is the softening point (in °C), lg P is the logarithm (base 10) of the penetration at 25°C (in 0.1 mm).

Other types of bituminous binders include hard paving grade bitumens, polymer-modified bitumens, bitumen emulsions, oxidised and industrial grade binders, cutback and fluxed bitumens. In Europe these bituminous binders are either specified or classified in accordance with harmonised European standards, as shown here:

- EN 12591—Specifications for paving grade bitumens (CEN, 2009a)
- EN 13924—Specifications for hard paving grade bitumens (CEN, 2014d)
- EN 14023—Specification framework for polymer-modified bitumens (CEN, 2010)
- EN 13808—Specification for cationic bitumen emulsions (CEN, 2013a)
- EN 13304—Framework for specification of oxidised bitumens (CEN, 2009b)
- EN 13305—Framework for specification of hard industrial bitumens (CEN, 2009c)
- EN 15322—Framework for specifying cutback and fluxed bituminous binders (CEN, 2013b)

14.3.1 Rheology

Rheology is the study of the relationship between force (stress) and deformation (strain) of engineering materials under a set of loading and environmental conditions. It is essential to understand how rheology applies to the characteristics of bituminous materials because of their viscoelastic behaviour, often represented as complex modulus and phase angle. Complex modulus is a representation of the viscoelastic response of a material under dynamic loading at a given strain level; it comprises viscous (loss) modulus and elastic (storage) modulus. Phase angle is the corresponding lag between the elastic and the viscous response.

As a thermoplastic material, the complex modulus of bitumen reduces (becomes softer) as the temperature increases, and vice versa (Fig. 14.1). It is broadly accepted that bitumen at a fixed rate of loading and at high temperatures will behave viscously, whereas at low temperatures, the response changes to elastic behaviour (Fig. 14.2). Higher values of phase angle indicate a tendency towards more viscous behaviour, whereas lower values indicate a more elastic response. This elastic behaviour (lower phase angle) indicates high stiffness and increased brittleness, whereas the viscous response (higher phase angle) reflects high ductility and low stiffness. Detailed discussions on the practical aspects of bitumen rheology as to the performance of bituminous mixtures are reported by Widyatmoko and Elliott (1999) and Widyatmoko (2003, 2004, 2005).

14.4 Bituminous mixtures

Road paving materials vary from bituminous (asphalt) mixtures, pavement quality concrete, hydraulically bound mixtures, unbound/granular and stabilised capping mixtures. This chapter specifically focuses on bituminous materials for use in surfacing and base layers.

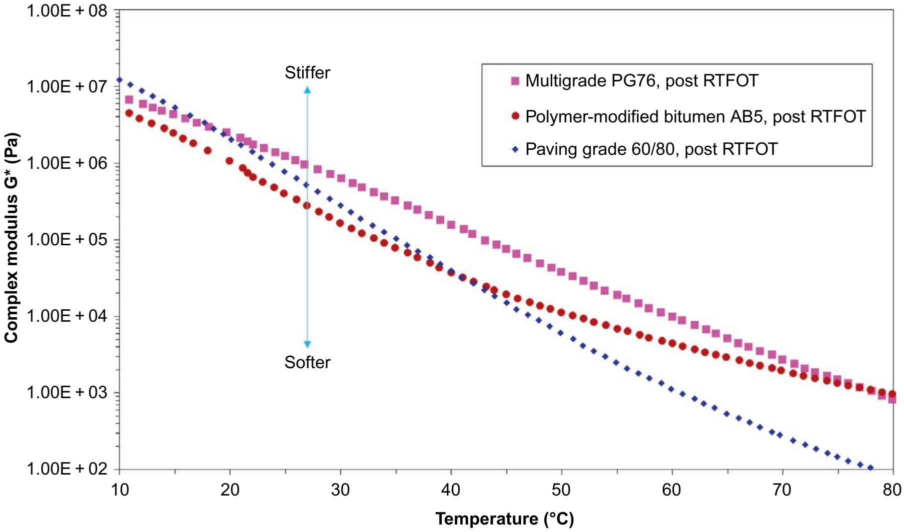


Fig. 14.1 Complex modulus as a function of temperature.

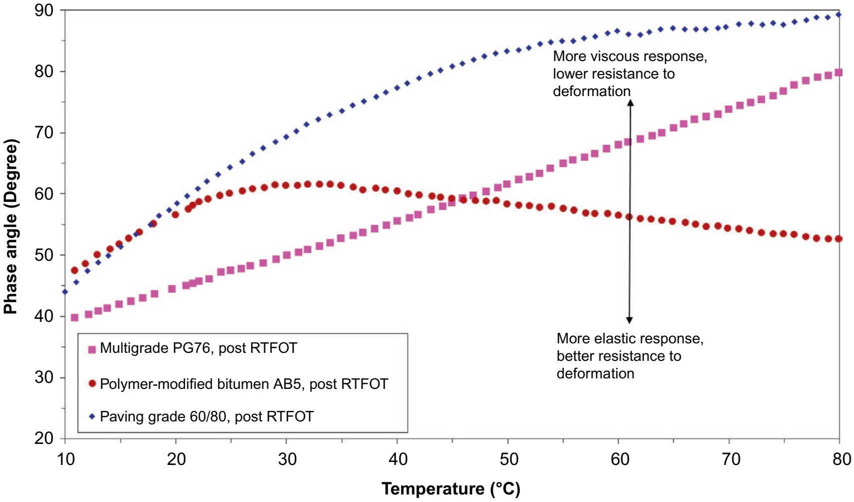


Fig. 14.2 Phase angle as a function of temperature.

Aggregates used in these mixtures are classified as primary and secondary aggregates. Primary aggregates are generally quarried from land or dredged from a sea or river bed, whereas secondary aggregates can be rejects (paste-coating, post-carbonation), by-products (slags, ash, dust), reused and recycled mixtures, construction and demolition waste, domestic waste and recycled asphalts and concretes. Case studies of the use of by-products and reprocessed aggregates in asphalts and concretes can be found elsewhere, for example, for the use of steel slag, refer to Ellis and Widyatmoko (1999); for the use of zinc slag, refer to Dunster et al. (2006); for the use of processed plastic, refer to Widyatmoko et al. (2007). For hot asphalt pavement recycling, studies on mixture design incorporating reclaimed asphalt pavement (RAP) materials were done by Widyatmoko and Elliott (2002), Widyatmoko (2008) and Carswell et al. (2008). Some of these case studies are discussed later in this chapter.

14.4.1 Types of bituminous mixtures

Bituminous mixtures, sometimes called asphalt mixtures, are blends of aggregates with different gradations, filler type and content and hardness with bitumen of different grades and quantities. Numerous types of bituminous material are designed to meet certain target mixture volumetrics (density and void content); some may incorporate performance-enhancing additives such as fibres and modified binders.

In the harmonised European Standards EN 13108 series, bituminous mixtures are classified in eight categories:

- EN 13108-1—Asphalt concrete (AC) (CEN, 2006a)
- EN 13108-2—AC for very thin layers (CEN, 2006b)
- EN 13108-3—Soft asphalt (CEN, 2006c)
- EN 13108-4—Hot rolled asphalt (HRA) (CEN, 2006d)
- EN 13108-5—Stone mastic asphalt (SMA) (CEN, 2006e)
- EN 13108-6—Mastic asphalt (CEN, 2006f)
- EN 13108-7—Porous asphalt (CEN, 2006g)
- EN 13108-8—Reclaimed asphalt (CEN, 2005)

AC, formerly known in the United Kingdom as dense bitumen macadam; HRA; and SMA are the most commonly used asphalt materials for road paving in European countries. The remaining materials within the EN 13108 series tend to be used on specialised applications such as bridges and airports.

AC materials are continuously graded, dense materials. In the United Kingdom, AC is typically used in binder course and base layers on trunk roads and motorways, whereas AC surface course has been specified for less travelled roads. High modulus base such as Enrobé à Module Élevé falls within this AC family. HRA materials are gap-graded dense materials, and the majority are generally of low stiffness and/or have low resistance to permanent deformation because of their high binder content and lack of design. However, these materials are also durable and are highly resistant to cracking. SMA materials rely on an interlocking aggregate skeleton and are always designed well because the exact mixture depends on the grading, shape and surface roughness of the aggregate. SMAs are binder-rich materials and usually need the addition of fibres and/or the use of a polymer-modified binder (PMB) to prevent binder drainage.

14.4.2 Production methods

Production methods for asphalt materials are generally classified based on mixing temperatures. Most of the bituminous materials used for paving roads were produced following HMA manufacturing processes. However, other manufacturing methods produce bituminous mixtures at lower production and installation temperatures; these

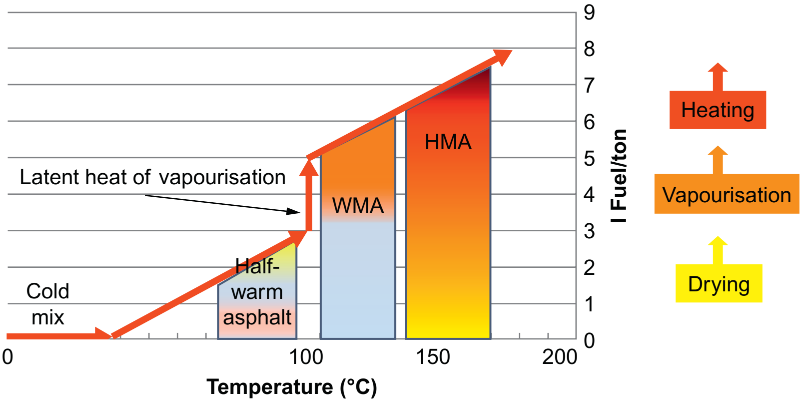


Fig. 14.3 Classification of asphalt mixtures by temperature range.
Source: EAPA, 2010. The use of warm mix asphalt. Position Paper, January.

methods of producing bituminous mixtures are known as low temperature asphalts (LTAs). Currently, there are three classes of LTAs: cold mix asphalt (CMA), half-warm mix asphalt (HWMA) and warm mix asphalt (WMA).

The European Asphalt Pavement Association (EAPA, 2010) defines WMA as material manufactured using special techniques that have a mixing temperature in the range 100–140°C. This compares with HMA at 120–190°C and HWMA at 70–100°C. In the United Kingdom, CMAs are defined as cold road materials where the aggregates are mixed at ambient temperatures, although other constituents such as foamed bitumen or bituminous emulsions may be well above ambient immediately before adding to the mixer. These classifications are illustrated in Fig. 14.3.

14.4.3 Specifications and design guide

Specification for Highway Works, SHW (Department for Transport, 2008a,b), and its Notes for Guidance are the most widely referred to documents for specifying bituminous materials for road applications in the United Kingdom. This specification is mandatory for use on the Highways England strategic network of roads (trunk roads and motorways) and has also been widely adopted by local Highway Authorities; only a small number of local Highway Authorities have their own specifications. For the design guide, refer to the United Kingdom Department for Transport Design Manual for Roads and Bridges Volume 7 (DMRB7) (Department for Transport, 1994 (1998)), which contains comprehensive methods and references for designing and maintaining road pavement.

14.5 Sustainability by design

Bituminous pavements are constructed using natural resources such as aggregates and binders. These resources are limited, and therefore their use should be designed with sustainability in mind, including minimising the use of materials through improved and efficient designs, maximising the service life of materials (improving performance and durability), minimising waste, maximising recycling, using recycled and secondary materials and using tools such as whole life costing, life-cycle analysis and ecological footprints. Old asphalt pavement removed during most road resurfacing and reconstruction projects (often called RAP) can be recycled economically into good-quality asphalt materials (Widyatmoko and Elliott, 2002), while conserving aggregates and bitumen, eliminating disposal problems, reducing transportation requirements and lowering fuel consumption and greenhouse gas emissions (Chappat and Bilal, 2004). Asphalt material is fully reusable and recyclable. A range of cold and hot asphalt recycling processes (which include cold in-plant recycling, full-depth cold in situ recycling with emulsion or foamed bitumen, hot in situ surface recycling, and hot in-plant recycling) are well developed and widely used around the world. In Australia pavement recycling has been reported to provide a service life up to 75 of that from a new construction, but at costs of 35–50% of that for a new construction (Verachtert, 1997). However, these advantages can be realised only when the recycled material can be shown to have an in-service performance and cost similar to the conventional material it replaces. With proper design and good quality control, recycled asphalt can be expected to perform as well as conventional materials (Widyatmoko, 2008). In addition to these recycling methods, over the past decade there have been an increasing number of initiatives to promote the use of sustainable materials in Europe. Some of these initiatives are not new but are improved techniques developed to reduce impacts on the environment, such as the use of rubber in asphalt, the feasibility for the use of synthetic asphalt binders produced from renewable sources and the introduction of LTAs.

14.5.1 Performance and durability

The important mechanical properties of a bituminous material with respect to pavement life are its stiffness, deformation resistance and fatigue characteristics. The stiffness of a material relates to its load-spreading ability, the deformation resistance relates to its ability to resist rutting during service, and the fatigue characteristics relate to its crack resistance properties. These properties are particularly important for the structural layers but also contribute significantly to the performance of the surface course. In addition, a bituminous material should be durable and have good resistance against age- and moisture-related damage during its designed lifetime. In this context, pavement durability can be defined as “the retention of a satisfactory level of performance over the structure's expected service life without major maintenance for all properties that are required for the particular road situation in addition to asphalt durability” (Nichols et al., 2010).

As with bituminous binders, bituminous materials exhibit viscoelastic behaviour in that they harden when subjected to low temperatures and fast loading times; conversely, they soften when subjected to high temperatures and slow loading times. Thus the stiffness of bituminous mixtures is highly dependent on temperature and loading time and closely related to stiffness of the respective bituminous binder. Fatigue cracking is among the main criteria for failure in the design of bituminous pavement. Normal traffic loading will result in fatigue unless there is very thick pavement, where the main failure would be either surface-initiated cracking or rutting within bituminous layers. There is a trade off between early-life performance of bituminous layers (such as resistance to rutting) and long-term durability (such as resistance to cracking and fretting). In this context, the main mode of distress should be predicted for each design.

Good bonding between pavement layers is desirable in order to provide a coherent composite structure with good resistance to repeated wheel loading by traffic. Weak bonding between bituminous pavement layers may result during construction or overlaying because of poor quality control, lack of tack coat, contamination of the lower layer or laying stiff materials in cold weather (Sangiorgi et al., 2002). However, bonding between bituminous layers may improve under traffic and environmental variations during early pavement life. Lack of bonding between pavement layers produces higher deflections under surface loading, because the layers act independently in the absence of shear continuity at the interfaces; this gives the same effect as a pavement with low material stiffness. If this is the case, cracks may propagate at a higher rate, leading to earlier pavement failure, as shown in Fig. 14.4.

14.5.2 Reuse and recycle

There is only a subtle difference between the terms reuse and recycle. Reuse of asphalt (bituminous) materials is often understood as adding RAP into new asphalt mixtures, with all the mixtures' components performing the same function as in their original application (such as adding RAP from an old asphalt layer to a new asphalt layer); this process can be traced back to the early part of the 20th century. On the other hand, recycling includes utilisation of RAP in different materials or functions, such as adding RAP to the foundation, fill or base material. However, in general, these two terms are better known as ‘recycling’, as defined by WRAP (Waste and Resources Action Programme—http://www.wrap.org.uk): recycling is “the reprocessing of wastes, either into the same material (closed-loop recycling) or a different material (open-loop recycling)”.

The first documented case of asphalt recycling, in the form of hot in-place recycling, was reported in the literature in the early 1930s using standard batch mixing equipment (Taylor, 1978, 1981). Taylor reported that the recycled surfaces provided

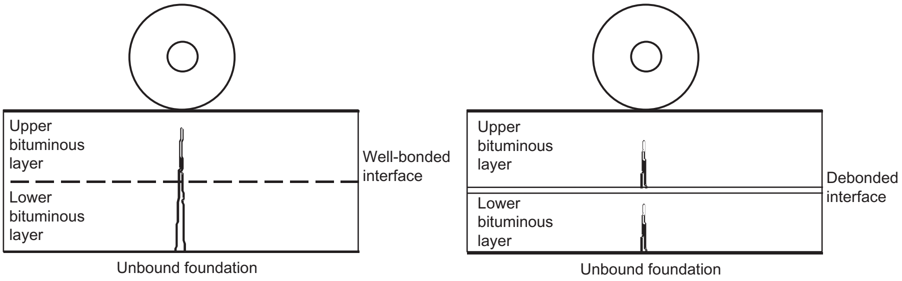


Fig. 14.4 Effect of layer bond to crack propagation.

satisfactory performance for 25 years. However, recycling was not common because of the increased cost. In the late 1930s the Road Research Laboratory in the United Kingdom was experimenting with a cold in situ recycling process, but implementation of this work was interrupted by World War II (Nicholls, 1996). However, the process was implemented following trials in 1948, and is still in use. This cold in situ recycling process is currently known as 'Retread'.

More significant developments in asphalt recycling technology and equipment did not occur until 20 years later when two events—specifically, the petroleum crisis of the early 1970s and the development and introduction in 1975 of large-scale cold planing equipment, complete with easily replaceable tungsten carbide milling tools—triggered the interest in recycling similar to the level of its worldwide use today.

At the end of the 1970s, hot in situ recycling was introduced in the United Kingdom with the importation of equipment and technology from the United States. Two processes of hot in situ recycling were developed, Repave and Remix, which are now recognized as standard rehabilitation treatments for asphalt pavements and are included in the UK Department for Transport DMRB7 and SHW 900 series Volumes 1 and 2, and Defence Estates Specification 27 (Defence Estates, 2008). Since the early 1980s, as reported at the 18th World Road Congress in 1987, recycling techniques have been used to provide a standard alternative material for both new construction and maintenance of roads in many countries around the world, including Australia, America and several of the Scandinavian countries. Since that time, the equipment manufacturing and construction industries have been proactive in the development of asphalt recycling methods and technologies, both of which have advanced exponentially in the past 25 years.

Recycling is done in different ways: recycling in situ (in place) or ex situ (in plant), using the reclaimed materials with or without adding new materials and keeping or changing the function and the characteristics of the materials. The choice depends on technical, environmental and economic issues. Approaches vary from country to country, depending on national needs, requirements, technologies, resources and so on. These factors have led to a research initiative to coordinate best practices in recycling road materials within the European countries, which was rolled out from 2009 to 2011 under a DIRECT-MAT (DIsmantling and RECycling Techniques for road MATerials) project (De Bock et al., 2011). In Jun. 2014 the EAPA published a position paper stating that 'Asphalt is the 100% recyclable construction product' (EAPA, 2014).

14.5.3 Retread

Retread is a cold process involving a preliminary assessment of the existing road to see if it is suitable for treatment. If the existing road construction is suitable, it is pulverized to a specified depth to produce a uniformly graded material. The surfacing layers are milled or pulverised to a depth of approximately 75 mm, broken up and levelled, and a bitumen emulsion is applied together with some virgin aggregates to further improve the grading; then it is re-harrowed and rolled before being completed with a surface dressing layer. Further applications of surface course treatments can be used to suit traffic loads. This process is also referred to as linear quarrying, because the primary source of the materials for this process are those recovered during Retread recycling processes, although a small portion of new aggregates are often added to correct the grading and target density. This approach may be a cost-effective solution to recycling and reusing distressed asphalt surface courses in urban and rural sites, on footways and on lightly travelled roads (Troeger and Widyatmoko, 2012).

In 2008 a separate assessment was carried out on Retread materials recovered from 11 sites located in Leeds (United Kingdom) that had been in service from 1–7 years. The selected sites were classified as state roads with relatively low heavy goods vehicle traffic loading, and the Retread layer was overlaid by either surface dressing or microsurfacing. The Retread materials constructed during the 1997–2005 period tested, as received, showed a relatively high range of variation in air voids but showed comparable performance with respect to stiffness, that is, in the 630–1050 MPa range; the stiffness range is not unusual for relatively voided AC incorporating 100 pen bitumen or softer. The recovered grading curves of the mineral aggregate and binder content were generally found to be consistent and within a narrow envelope. The recovered binder data showed that binder hardening occurred at a greater rate during the first 4 years in service and then stabilised during the next 6 years in service. Some deviations from the expected changes in binder properties in service (decrease in penetration, increase in softening point and increase in stiffness) were observed for the Retread materials constructed in 2006 and 2007, consistent with the different mineral aggregate gradings. Overall, these results suggest that the Retread process is able to restore the mechanical properties and the profile of a distressed surface course, extending the life of surfacing on lightly travelled roads.

14.5.4 Repave

Repave involves a process of heating the top 25–35 mm of the existing surface course, scarifying and re-profiling by adding new material (typically, with a target of additional thickness around 20–25 mm) immediately on top of the scarified layer, followed by compaction of both the scarified and new material in one pass (Fig. 14.5).

In this process, the new layer comprises a composite between recycled and added materials. The main benefit of this process is that it can be applied to all classes of roads and airfields provided that the asphalt surfaces and substrates meet certain conditions. Although the surfacing may have some moderate signs of age hardening, the substrate should be in sound condition. In the United Kingdom, the two main specifications and design guides are Clause 926 of SHW and DMRB7 HD 31/94. The benefits of the process are obvious: compared to a conventional surfacing which planes 40–50 mm of existing material off the surface and takes it off-site, the Repave process takes away at maximum only half of this amount. On roads where the conditions allow, only a channel plane might be carried out, reducing the waste to an absolute minimum.

The following is a case study of a project which shows how the Repave process significantly saved cost. In Mar. 2011 resurfacing 33,000 m^2 of worn out and partly cracked surface course on the A64 Malton Bypass in the United Kingdom was completed using the Repave process. This work was originally planned in two phases because of budgetary restrictions. However, by using the Repave process, the work on the whole project was actually completed in one phase and within the available

Fig. 14.5 Repave process: heating, scarifying and compaction.

budget. Data recorded for this project shows that the Repave process was 37% cheaper and was delivered 20% faster than a conventional method, taking into account all aspects, including process cost, traffic management and road lining.

14.5.5 Ex situ recycle

Recycled materials must perform similarly to conventional materials as well as demonstrate cost-effectiveness in order to be acceptable. Where existing pavements have performed satisfactorily, it is likely that the original aggregates will comply with the standard requirements, although some degradation can be expected on reclamation. Blending of new aggregate will probably be necessary in order to meet grading requirements. Bituminous binders tend to harden during the mixing process and with age, which necessitates the addition of a softer binder to restore the equivalent properties as required under the original specification.

Consistency of the RAPs to be recycled is of paramount importance because variable materials will cause problems with the control of quality and may hinder the efficiency of the recycling operation. In this context, ex situ recycling offers better control on the consistency of the RAPs because the process allows stages of assessments from the stockpile, additional screening and production of the recycled asphalt mixtures similar to how primary aggregate supplies are dealt with. Sufficient and consistent sources of suitable material for the scheme being considered need to be identified either in existing pavements from stockpiled planings of known origin or from another suitable source. The assessment of the properties of the existing material proposed for recycling can be made using cores sampled from the carriageway or from samples taken from stockpiles in accordance with current practice. The frequency of sampling should be sufficient to determine how consistent the reclaimed material is and to provide representative samples for composition analysis and measurement of the properties of recovered binder.

A flow chart for assessing the suitability of reclaimed materials and the proportion to be recycled is shown in Fig. 14.6.

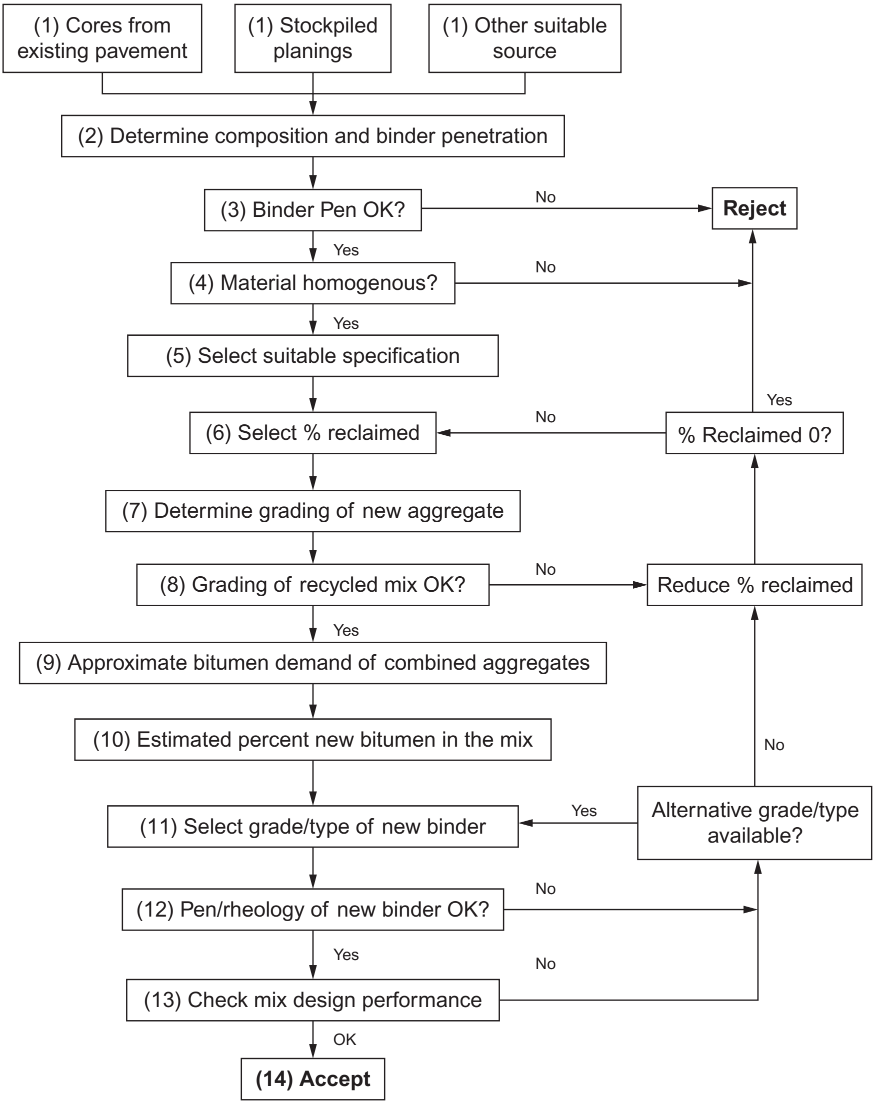


Fig. 14.6 Assessment of suitability and proportioning of RAP for hot-mix ex situ recycling. Source: Widyatmoko, I., 2008. Mechanistic-empirical mixture design for hot mix asphalt pavement recycling. Constr. Build. Mater. 22 (2), 77–87, February, Elsevier.

The UK SHW Clause 902 allows RAP material to be used in the production of asphaltic surface course, binder course and base with a maximum amount of 10% in surface course and 50% in all other layers with additional performance required when the recycled content exceeds 25% by mass. The SHW Clause has allowed manufacturers to adjust their feedstock and optimise production and performance within their existing manufacturing plants. This approach is not dissimilar to that of Superpave mixes, where the amount of RAP allowed typically ranges from typically 10–15% for surface courses (except in the highest volume highways) and up to 50% for other layers (McDaniel and Nantung, 2005). In practice, conventional batch plants (with suitable modifications) can handle up to 20% RAP; above this level, preheating of the RAP may be required. Drum mix plants, on the other hand, can handle up to 70% RAP. However, these trends are expected to change because new asphalt manufacturing technology has emerged which claims a capability of incorporating more than 90% RAP into the recycled asphalt mixtures; this can be done by introducing workability-enhancing additives or rejuvenating agents into the mix design or with the use of warm mix technology (Howard et al., 2013). The following is a case study related to recycling of high-performance asphalt into thin asphalt surfacing reported by Carswell et al. (2008).

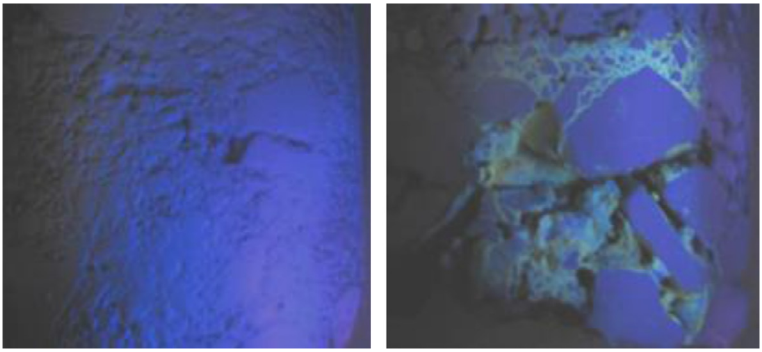
Thin asphalt surfacing requires substantial amounts of high-quality aggregate; therefore using reclaimed asphalt in thin asphalt surfacing will help promote sustainability of premium aggregates. The first applications of thin surfacing products in the United Kingdom took place 15 years ago, and sites with these surfacing products will increasingly need to be replaced. As a result, the old material will become available for recycling. The need to recycle thin surfacing systems as part of efforts to promote sustainability within road construction is more critical than with many other generic surfacing materials because of the quantity of relatively scarce aggregates with high skid-resistance properties within the layer. Trial sites were successfully constructed on the access road to the Renishaw asphalt plant near Sheffield and on the strategic road network at A1(M) Hatfield and A405 Bricket Wood (United Kingdom), including the use of PMBs and up to 30% RAP in the mixed asphalt. These trial sites were assessed for visual condition, density and texture depth, as well as samples of material for determination of grading, recovered binder and resistance to permanent deformation. Laboratory investigation of aggregate suitability and calculations of maximum theoretical amounts of RAP that can be added to new surface course materials generally showed that one of the limiting factors was the capability of the local asphalt plants to add high quantities of RAP to the mix. The findings demonstrated that 10% RAP can be added easily to new materials without affecting grading. As the proportion of RAP increases up to 30%, greater care needs to be taken on assessing grading compatibility and on how to treat the residual binder present in the RAP as a proportion of the ‘active’ binder content in the recycled surface course layer. To date all of the trials are performing well, although some problems occurred during construction which were caused by factors other than the inclusion of RAP. Even though the feasibility of the process has been demonstrated, further investigation is needed about the potential effects of using recycled materials, with regard to where and how asphalt planings from various sources and of various layers need to be stored in order to avoid contamination, thereby maximising the use of aggregates with high polished stone value. This work led to the publication of Road Note 43, a best practice document on recycling surfacing into thin surfacing (Carswell et al., 2010).

14.5.6 The 'Tar' matter

Tar is a big problem when it comes to its removal from roads and for recycling. RAP that contains tar is categorized in the EU legislation as a dangerous waste. This classification is based on the high polycyclic aromatic hydrocarbon (PAH) content present in coal tar and the proven carcinogenicity of the product. Tar is no longer used in many European countries, but it may be found in old pavement layers. Consequently, a quick method is needed to detect tar onsite and to carry out safe procedures to manage the problem.

A quick method to detect tar onsite is by using a tar spray test. The tar spray test is a rapid, qualitative indicator of the presence of polyaromatic compounds (PACs) typically found in tar. PACs also exist in other road construction materials (eg, bitumen and cutbacks like kerosene) but at low concentrations. Fig. 14.7 shows tar- and non-tar–containing samples under UV light as they appear following the application of a tar spray test; yellow marks denote a positive result, which means a possible presence of tar. In this test, the probability of obtaining a false positive result in the tar spray test with such materials is low, and a positive result in the tar spray test is a strong (but not definitive) indicator of the presence of tar. For quantitative results, this test should be considered in conjunction with the results from other tests (ie, total PAH by Gas Chromatography-Flame Ionisation Detection (GC-FID)).

There is a complex environmental regulation that tar-containing material be subjected to removal or transported off-site for further treatment (Environmental Agency, 2005; Wilkinson and O'Farrel, 2009). Furthermore, EN 13108-8 excludes the use of reclaimed asphalt-containing tar. When it comes to pavement rehabilitation work, the


Non-tar-containing sample

Tar-containing sample

Fig. 14.7 Tar- and non-tar-containing samples under UV light.

less complicated solution is to leave the tar-containing layer in place or, if improved performance is required, to treat the tar in situ. Because tar is susceptible to leaching and can easily release hazardous fumes when in contact with heat, any treatment must be carried out at a low temperature, which can be done in situ to encapsulate the tar-containing RAP, such as by utilising bitumen emulsion, foam bitumen or cement stabilisation. These treatments are widely accepted as environmentally acceptable methods for reusing tar-contaminated road material.

14.5.7 Recycle with foam bitumen

Producing recycled asphalt using foam bitumen technology allows mixing of RAP with unheated aggregates and is seen to be more environmentally acceptable than using a hot mix process. Foaming technology was first introduced by Prof. Ladis Csanyi in 1957 and then refined by Mobil Oil (Australia) in the 1960s by creating an expansion chamber. After Mobile Oil's patent on the foamed bitumen technology expired in 1991, many systems were developed worldwide.

In the mid-1990s, the equipment manufacturer Wirtgen further developed this system by creating the Wirtgen WLB-10 laboratory foaming plant in which both air and water are injected into hot bitumen in an expansion chamber. This process involves injecting a foaming agent into hot penetration grade bitumen, resulting in the bitumen expanding 10–15 times its initial volume, which is then sprayed through specially designed nozzles onto, and mixed with, cold moist aggregate (Wirtgen, 1998). In the expanded state, water is mostly lost as steam, and the foam eventually consists of 95% air by volume. As the air escapes and the foam collapses, it leaves a bitumen residue with properties similar to the base bitumen. The increased volume and surface energy of the foam promote coating and mixing of the aggregate. The bitumen used for foaming can normally range in penetration grades from 50 to 200. Higher penetration grades tend to foam better while the lower grades produce materials of higher stiffness; generally, grade 100 is considered suitable. Typical bitumen contents for foamed bitumen mixtures range between 2% and 6% and are mainly dependent upon the aggregate type and the fines content. Too little foaming water results in inadequate foam dispersing in the mixture, while an excess of water causes difficulty in compacting, thus reducing the strength of the compacted material (Sunarjono et al., 2007; Widyatmoko and Sunarjono, 2007). For the foamed bitumen treatment, however, the material should have a certain proportion (between 5% and 20%) passing the 63 μm sieve size. If this is not the case, the grading should be adjusted by importing and spreading a layer of suitably graded aggregate or filler such as lime or fly ash over the surface of material to be recycled; Portland cement or another hydraulic binder is often added to accelerate curing. The foamed bitumen is generally less expensive than bitumen, and it is easily used with little risk of cracking from shrinkage. The balance between foamed bitumen and some hydraulic binders can be a compromise between early-life performance and the risk of thermal cracking. The recycled pavement can be designed to gain strength rapidly, and provided that the density achieved is at least 95% refusal, it can be opened to traffic immediately after compaction. The coating of the aggregate with the foamed bitumen may be carried out either in situ or in tailor-made mixing plants (in-plant).

The in situ process, also known as ‘Foamstab’, consists of recycling a distressed pavement by milling the road to a certain depth (100–300 mm) using a heavy duty rotovator. During a second pass of the rotovator, the pulverised pavement is injected with foamed bitumen. The bitumen is continuously supplied to the rotovator from a road tanker, and the two vehicles move in tandem along the site. Small quantities of cementitious filler (cement, lime or fly ash) are normally added to assist the adhesion of the bitumen to the aggregate. After mixing, the material consists of coated fine aggregate and partly coated coarse aggregate. Fine bitumen filaments are interspersed throughout the mixture. The mixed material can then be moved, levelled and shaped. The Foamstab process can be used to recycle distressed asphalt pavement and granular base and sub-base layers. The in-plant mixed process produces a material known as Foamix. The plant in the United Kingdom originally licensed to produce Foamix was a conventional hot mix plant with the addition of a cold-mix production stream. However, progress in the production technology for Foamix has led to the development of mobile mixing plants which produce a material that is best described as site-mixed rather than plant-mixed. The feed aggregate for Foamix may be virgin aggregates, road planings, marginal construction materials or combinations of these. Foamix is produced in a cold condition at output and has a similar appearance to material produced in situ (Foamstab), in regards to the distribution of coated fines and partly coated coarse aggregate. Normally, the material remains useable for an extended period of time (up to 3 months), depending on weather and stockpile height. The storage of foamed mixture materials will not change the handling characteristics provided that there is no significant buildup of inter-particle pressure and no loss of moisture; however, the overall mechanics of mixture storage are not fully understood (Biczysko, 1997).

In 1986, Lunedale Road in Dartford (Kent) became the first road in the United Kingdom to be recycled using foamed bitumen as a binder (Hubert, 1988a; Energy Efficiency Office, 1989). The Lunedale Road project demonstrated that recycling with foamed bitumen under United Kingdom conditions could produce a consistent material with good compaction properties. Deflection and deformation results indicated that a recycled pavement with a foamed bitumen base 205 mm thick and topped with a regulating course of 15 and 40 mm of HRA surface course material would not need structural maintenance in the subsequent 20 years as a result of predicted traffic loadings up to 0.5 msa (millions of standard axles) (Hubert, 1988b). This result was comparable to a predicted life of 0.33 msa for a pavement constructed with a 200pen DBM base and a binder course 205 mm thick.

In South Africa several projects using foamed bitumen have been completed since May 1994 and have provided useful experience. Investigations were carried out in 1997 by the CSIR's Division of Roads and Transport Technology, including deflection measurements utilising a falling weight deflectometer and resilient modulus testing using core samples. The results confirmed adequate performance of the recycled pavements. The road pavements treated (in 1994 and 1995) with foamed bitumen in the base layer are presently performing well under traffic, and investigations indicate that they should achieve their anticipated design lives (Lewis and Lindsay, 1998).

Foamix was used as base layer during construction work on the M62 from junctions 5 through 6, which is one of the major motorways in the United Kingdom. The material was reported to have achieved an average ITSM of 5400 MPa, thus providing a design life identical to a conventional heavy-duty base and a £50,000 savings in the construction cost. Full-depth 200 mm layer thickness recycling was adopted during construction of the A21 Sevenoaks Bypass in the United Kingdom, which involved the use of foam bitumen (3%), lime (2%), cement (2%) and PFA (6%) blended with RAP (87%). This process reported a saving of 30% of the project's total construction budget, which was equivalent to a direct savings of £525,000, when compared to cost of using conventional asphalt (SMART, 2004). In addition to a similar level of cost-saving (29%), up to a 30% reduction in construction time was reported for a pavement rehabilitation job that involved in situ recycling of a primary road in Portsmouth, England, where a foamed bitumen bound recycled mixture was used to replace the existing bound layers and part of the sub-base. The road has shown various defects and has suffered from poor riding quality; after the rehabilitation work, the new pavement was expected to carry 60 msa.

14.5.8 Low temperature asphalts

European countries are using technologies that appear to allow a reduction in the temperatures at which asphalt mixes are produced and placed. These technologies are known as LTAs comprising WMA, HWMA and CMA. The immediate benefit of producing LTAs is the reduction in energy consumption required by burning fuels to heat conventional asphalt mixtures to temperatures in excess of 160°C at the production plant. These high production temperatures are needed to enable the bituminous binder to become viscous enough to completely coat the aggregate in the asphalt mixture, to have good workability during laying and compaction, and to maintain durability during exposure to traffic. The decreased production temperature brings the additional benefit of reduced emissions from burning fuels, fumes and odours generated at the plant and the paving site.

A number of proprietary technologies have been developed and used in the European and North American countries to produce LTAs; these are summarised in Table 14.2.

The preceding technologies allow the production of LTAs by reducing the viscosity of the bituminous binder at a given temperature. This reduced viscosity enables the aggregate to be fully coated at a lower temperature than what is traditionally required in asphalt production. However, some of these technologies require significant equipment modifications. This technology could have a meaningful impact on transportation construction projects in and around non-attainment areas such as large metropolitan areas with air quality restrictions. The reduction in fuel usage to produce the mix would also have a significant impact on the cost of transportation construction projects.

Fig. 14.8 illustrates how low the production and compaction temperature can be by using the wet sand technique. Specifically, the HWMA was mixed at temperatures as low as 90°C, and then discharged to delivery lorry at 83°C, and the compaction was

Table 14.2 **Proprietary low temperature asphalt systems**

Technology	System	
Zeolite additives	Advera	Aspha-Min
Organic (wax) additives	Asphaltan A Romonta N	Asphaltan B
	Sasobit	Sasolwax Flex
Fatty acid derivative additives	Hypertherm	Licomot BS 100
Chemical additives	Cecabase RT Ecoflex or 3ELT	Evotherm DAT
	Evotherm 3G or REVIX	Qualitherm
	Rediset WMX	Sübit VR
Other specified additives	Thiopave	TLA-X
Emulsions	ECOMAC	Evotherm ET
Foaming technology	Accu-shear dual warm mix additive system	Aquablack WMA
	Adesco/Madsen Static Inline Vortex Mixer	AquaFoam
	Double Barrel Green Eco-Foam II	LEAB
	HGrant Warm Mix System	LT Asphalt
	LEA (Low Energy Asphalt)	NA Foamtec
	Meeker Warm Mix Asphalt system	Ultrafoam GX
	Warm Mix Asphalt system	
Other processes	Low Emission Asphalt	WAM-Foam

Source: Nichols, J.C., 2015. Low temperature asphalts. Asphalt, Pavement Engineering and Infrastructure Conference, Liverpool John Moores University.

Fig. 14.8 HWMA production using wet sand technology.

completed at surface temperatures closer to 70°C. As shown by these pictures, barely any fume was generated during the discharge and construction processes. The benefits of these technologies in terms of energy savings and air quality improvements are promising, but these technologies need further investigation and research in order to validate their expected performance (specifically long-term performance and durability) and added value.

14.6 Preservative maintenance

Pavement surfacing, whether it is new or old, would be expected to deteriorate and continue losing serviceability under the prevailing traffic loading and environmental conditions, and preservative maintenance and repair are usually required as serviceability falls below a certain level. Fig. 14.9 presents an idealised scenario where the condition of road surface will continue to deteriorate without preservative maintenance (shown by the red curve) and another scenario where the condition of the road surface can be restored to the previous serviceability level with preservative maintenance (shown by the green curve).

Decisions on the maintenance strategy for when remedial work should take place is in most cases considered critical because the cost increases as the structural integrity decreases. However, the decision about when this 'critical condition' has been reached is rather complicated because pavement deterioration may involve various types of distress, which can be functional or structural in nature, and other factors which contribute to the deterioration (such as traffic, environment and material properties) may also be involved.

Pavement preservation represents a proactive approach to maintaining existing highways; this includes routine maintenance, preventive maintenance and minor remedial work. A number of definitions are associated with pavement preservation; for

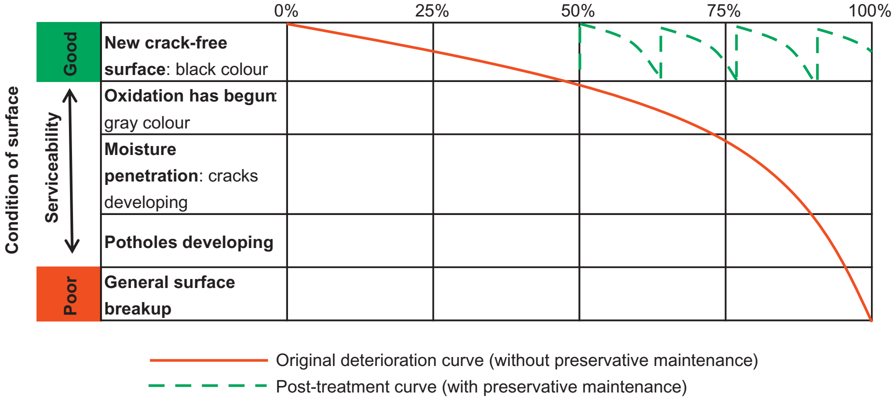


Fig. 14.9 Idealised presentation of surface condition over time, with and without preservative treatment.

example, the Federal Highway Administration (FHWA) Pavement Preservation Expert Task Group in the United States has defined it as "a program employing a network level, long-term strategy that enhances pavement performance by using an integrated, cost-effective set of practices that extend pavement life, improve safety and meet motorist expectations". In the United Kingdom suppliers of pavement preservation materials joined together under the umbrella of the Road Surface Treatments Association (RSTA) and formed an informal group known as the Pavement Preservation User Group (PPUG); this group recently changed its name to Asphalt Preservation Sub-Committee. They have considered three levels of preservative maintenance:

- Preservative treatments—Aimed at extending the defect-free life of an otherwise sound surfacing (age 1–5 years).
- Rejuvenative treatments—Aimed at extending the life of surfacings which are already weathered but substantially intact (age 5–10 years).
- Restorative treatments—Aimed at the stabilisation and rejuvenation of surfacings.

Due to oxidation and to the direct influence of sun, salts, acids and oils, the bitumen in the asphalt surface of a road ages rapidly. At the same time the road surface is subject to increasing heavy mechanical forces from the passing traffic. As a result the bitumen loses its flexibility and adhesive capacity and it becomes brittle. Many suppliers of preservation materials for pavements have used the preceding statements to present their business case that, in order to preserve their service life, road surfacings require treatment (preservation) after 3–5 years of traffic, and repeated applications are recommended to further extend the service life.

A number of pavement preservation systems marketed commercially in the United Kingdom, including some with the approval of the Highway Authorities Product Approval Scheme (HAPAS), claim to preserve and extend the life of asphalt surfacing. The predominant way these systems are assumed to work is either by the rejuvenation of the binder through modification of its chemical composition or by retarding the ageing process of the binder constituent of the asphalt mixture. All the processes involve the application of a liquid agent to the existing asphalt. Some include mixing of the agent with the asphalt in situ. Such products, in principle, have the potential to extend service life at a reduced cost, thus preserving the pavement assets.

It is claimed that these rejuvenators can penetrate into the existing surface to a depth of up to 12 mm, rejuvenating, binding, sealing and consolidating it; however, data from Vallerga (1963) suggests that the effective depth of rejuvenation was only up to 9 mm. Furthermore, based on the work on the hard shoulder of US 99 at Lodi in Aug. 1961, Vallerga found that only cationic 'maltene' rejuvenator penetrates and more effectively restores the penetration of the aged bitumen, whereas bitumen emulsion appeared to have less effect. Brownridge (2010) reinforced Vallerga's view that, in order for a rejuvenator to penetrate, it cannot be retarded by blending in a bitumen emulsion or be formulated into a quick dry emulsion because the effectiveness will cease as soon as the absorption stops (ie, the emulsion has cured). Many bitumen emulsions being marketed claim rejuvenation capability; however, Browridge stated that if "the (bitumen) emulsion breaks or cures on the pavement surface then it is sealing, not rejuvenating". He further argued that engineered cationic emulsions containing maltene saturates (light fractions), which are wax free, should be used as the base medium for rejuvenators. This engineered emulsion would be better able to penetrate ('diffuse') into the bitumen film that is being rejuvenated, because of its solvency effect with the binder; the molecular composition of the maltene base oil used in the formula provides this solvency without the use of distillates or solvents. In brief, the following criteria are widely adopted as measures of the effectiveness of a bitumen rejuvenator, specifically its ability to

- Restore the binder rheology, for example, increase the penetration value, reduce viscosity or reduce stiffness in the top portion of the pavement surfacing where rejuvenator was applied.
- Seal the pavement against ingress of moisture and air, reducing the risk of stripping and/or slowing down oxidative hardening of the upper layer and below.
- Improve the durability and extend the service life of the treated surface course due to any or a combination of the preceding factors.

However, Widyatmoko et al. (2012) reported that the effectiveness of preservative treatments varied with the type of preservative, such as whether it was solvent or emulsion based, and the post-treatment ageing characteristics. Some preservative agents appeared to show only short-term effectiveness of the treatments before subsequent, rapid age-hardening took place, whereas other preservatives seemed to maintain the effectiveness of the treatments for much longer. For the former, reapplication of the treatments would be necessary after 3–5 years, whereas the latter treatments could be reapplied after 5–10 years, as illustrated in Fig. 14.10. Findings from this work highlight the importance of carefully selecting preservative treatments to suit the maintenance plan for each road asset, as well as considering the frequency of reapplying treatment.

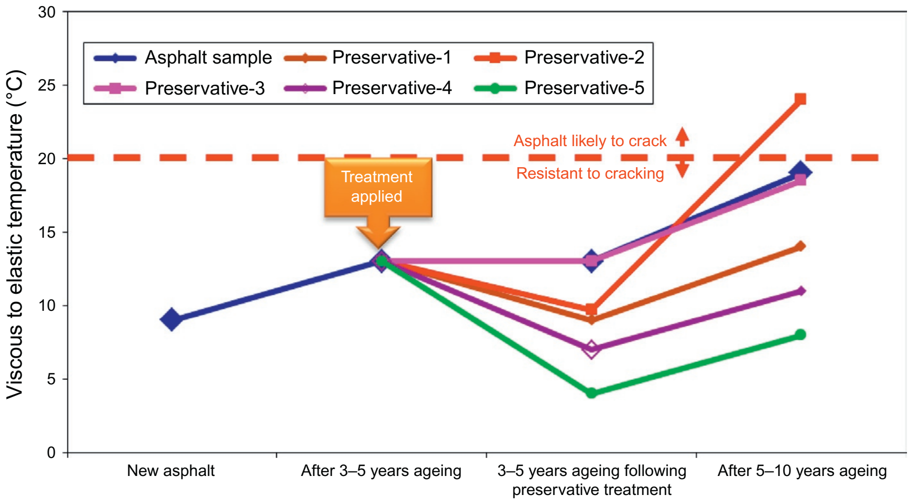


Fig. 14.10 Effectiveness of preservative treatments.
Source: Widyatmoko, I., Elliott, R., Heslop, M., Burton, D., 2012. Assessment of the effectiveness of asphalt pavement preservative materials. Euroasphalt Eurobitume Congress, 13–15 June, Istanbul.

14.7 Future trends

The requirement to build more sustainable and low maintenance road infrastructures has become very important because of the expected increase in road construction around the world and its impact on the environment. This goal can be achieved by the use of good practices during the design stage and during the preservative maintenance programme. Good practice in the pavement design includes the importance of providing good rideability and safety characteristics, good bonding between layers and durability. In addition, extending the serviceability of road pavements can be done by having preservative maintenance programme in place during the service life of the road pavements.

To promote sustainability, road pavements should, wherever possible, use materials from sustainable sources or production methods. As the technology matures with case studies and success stories collated from around the world (many from the United States, South Africa and Europe), LTAs are gaining momentum to attract attention from authorities and designers as a sustainable alternative to conventional hot mix paving. There are well-established mix design methodologies and production technologies on the use of these materials, including best practice guides of their applications on full-scale projects; many of these were presented in this chapter. In 2014 the National Asphalt Pavement Association reported an encouraging trend in the use of RAP as road construction materials in the United States; a summary is illustrated in Fig. 14.11 (Hansen and Copeland, 2014).

Amongst the data presented in Fig. 14.11, the total weight of WMA was estimated at 106.4 million tons, which was over 30% of the total asphalt production in 2013; this

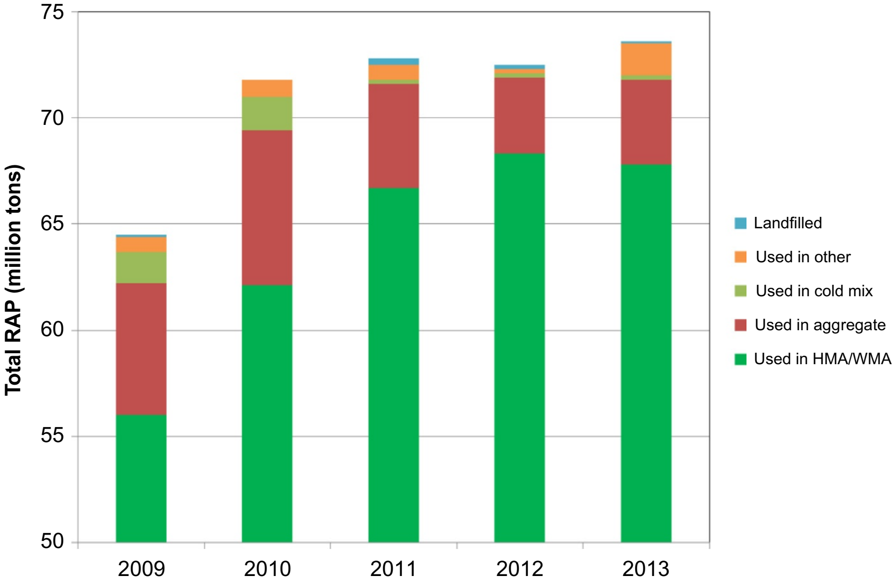


Fig. 14.11 The use of reclaimed asphalt pavement (RAP) in the United States.

was a 23% increase over that figure in 2012. It was also reported that more than 87% of WMA were produced by foaming technology (Hansen and Copeland, 2014).

Trends also show an increase use of sustainable material in the United Kingdom; however, the national specifications have yet to keep pace with these continuously improved technologies. For example, the use of RAP is covered in the current asphalt standard (BS EN 13108-8) and in the United Kingdom's design guide and specifications such as DMRB7 (1998) and SHW (2008). However, these documents are biased towards their use in hot mix asphalt, cold mix or foam mix and lack guidance on the applications of other types of WMA and HWMA. The latter means that their use will be treated as a departure from the specification which will be subject to a series of additional approval procedures, or treated as proprietary materials.

UK specifications (SHW series) allow the use of proprietary materials, which is currently covered under HAPAS. In this context, these materials must satisfy the requirements dictated by formal approval schemes, which will be in accordance with the relevant British-adopted European Standard (such as HAPAS) or the European Technical Approval and the Construction Products Regulation (Regulation (EU) No 305/2011, see https://osha.europa.eu/en/legislation/directives/regulation-eu-no-305-2011-construction-products, last accessed 9 Sep. 2015). However, there have always been questions about design assurance and production control of these products; these questions are raised mostly because of limited experience in using the relatively new technologies.

The preceding will challenge the industry and authorities to work together to accommodate the increasing demand on the use of sustainable materials and on the use preservative maintenance programmes with updates on the regulations and approval systems.

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Sustainability of cement, concrete and cement replacement materials in construction

15

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15.1 Introduction

15.1.1 Concrete, a popular material

Sustainable development is a process which enables all people to realise their potential and improve their quality of life in ways that simultaneously protect and enhance the Earth's life-support systems. Sustainable development involves maintaining our current rate of development, while leaving suitable resources behind for later generations to continue to develop (Sonebi, 2015; United Nations, 1987). The preservation of the environment is one of the principal features of sustainable development with a focus on reducing greenhouse gas (GHG) emissions. The use of concrete creates a large part of worldwide CO_2 emissions (Kline and Barcelo, 2012). Concrete's thermal mass proves to be a good feature of the material. Using this advantage, a well-designed concrete structure can make the difference in terms of energy efficiency during its service life. So, if concrete is not environmentally friendly in the manufacturing process, it does remain an excellent means for achieving energy efficiency and therefore for reducing global CO_2 emissions.

Moreover, the Kyoto Protocol has engaged several countries in trying to reduce their emissions. To reach this objective, it is essential that the concrete industry achieve substantial improvements in terms of reducing CO_2 during the production of cement. To target those enhancements and to know their potential scope, it is important to be able to assess the carbon footprint of concrete. This process will also give rise to questions about the pertinence of calculating and implementing a from cradle-to-grave footprint, that is to say, by analysing the greenhouse emission gas for all 'life cycle stages from the extraction or acquisition of raw materials to recycling and disposal of waste' (Kline and Barcelo, 2012).

Since the start of the industrial production of cement in the mid nineteenth century, concrete has become the world's single most widely produced man-made material, with an estimated annual production of 10 km^3 (Gartner, 2011). Concrete has various construction applications, including residential and nonresidential buildings, bridges, pavements, and dams (Table 15.1). Concrete is used for new construction in order to support the growing worldwide population and for the repair and maintenance of existing constructions (more than 50% in the European context, Table 15.1).

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Table 15.1 Use of concrete in the European context (CEMBUREAU, 2014)

	Construction in Europe Break down by type in 2014 (%)
New civil engineering	13.6
New residential	17.4
Residential repair and maintenance	27.8
New nonresidential	16.7
Nonresidential repair and maintenance	15.9
Civil engineering repair and maintenance	8.6
	100

Table 15.2 Cement roadmap made in 2009 cited by Schneider et al. (2011)

	Demand scenario	Production (Gt cement)
2006	Low/high	2.54
2015	Low	3.36
	High	3.48
2030	Low	3.34
	High	3.86
2050	Low	3.68
	High	4.38

The estimation made in 2009 by the International Energy Agency for the long-term demand for cement has already been exceeded (between 3.36 and 3.48 billion tonnes in 2015) with an estimated annual worldwide cement production in 2014 of 4.3 billion tonnes, that is, an increase of 6.7% from 2013 (Table 15.2). This increase is being led mostly by the strong demands in emerging economies such as those in China, India other developing Asia countries (CEMBUREAU, 2014).

Following the recent strong environmental initiatives led by several countries, the concrete industry is working hard to reduce its environmental impact and to change its image of being a dirty industry. For example, considerable efforts are being made at each construction phase to enhance the performance and durability of concrete, thus producing sustainable buildings. Furthermore, the cement industry is updating its production lines and implementing new burning processes in order to limit the CO_2 emissions and energy consumption and is producing new types of cements, including blended cements with mineral admixtures. These are some alternatives that are being gradually introduced into the market to enhance the sustainability of concrete. Concrete producers are innovating by using new concrete types, such as high-performance concrete (HPC) (Sonebi and Khayat, 2001), self-compacting concrete (SCC) (Sonebi, 2004; Sonebi and Bartos, 1999, 2002) thanks to the use of supplementary cementitious materials (SCMs), mineral additions and the development of chemical admixtures to enhance specific properties of concrete (Sonebi, 2015). In addition, project owners are adopting new habits by choosing the use of appropriate materials in the early concept or scheme phase. Nevertheless, appropriate and sustainable materials are not yet among the major criteria in their decision making. Based on a study conducted by the Arup and World Business Council for Sustainable Development (WBCSD) (2012), concern for sustainability generally comes after concern for factors such as technical performance, project delivery and supply of materials.

15.1.2 Strength and weakness of concrete

Concrete is a heterogeneous material obtained by mixing solid components, including aggregates, cement and mineral additions with water. The produced material is a flowing mixture which can be used and adapted to cast structural elements with different shapes and geometries. After a rest period (usually a few hours or few days), the concrete develops its strength, hence producing a solid material which, depending of its design and durability, can last for several decades. The popularity of concrete can be attributed to its economical aspect, which is based on its low energy consumption and low component cost, its abundance in local materials as well as its easy production and use. Concrete has various advantages compared to other materials, including versatility, mechanical performance, durability, and so on. One advantage relates to the two states of concrete:

- The fluid state of fresh concrete allows ease of transport, ease of its production at a construction site, ease of putting it in place, as well as significant versatility and architectural liberty, thanks to new technology such as HPC and SCC
- In the hardened state, concrete presents excellent mechanical properties and great durability

Secondly, concrete is a low-cost material because of the low cost of its primary materials, especially aggregates and water, which account for more than 80% of its volume; and because of the possibility of local production which reduces the cost of transportation. Concrete can even be produced directly on big construction sites.

From an environmental point of view, the major drawbacks of the use of concrete are due mainly to their huge success, which leads to:

- high consumption of natural resources such as raw materials for cement production, aggregates and steel for reinforcement
- high energy consumption needed for the production of cement and reinforced steel
- increased emissions of GHG, mostly carbon dioxide (CO_2), mainly because of cement production and energy used in the maintenance of buildings
- huge volume of construction and demolition waste generated at the end of service life of structures

15.1.3 Why concrete needs to become 'greener'? Why CO_2 emissions need to be reduced?

The greenhouse effect, a natural phenomenon, refers to the atmosphere's capability to prevent part of the sun's radiation from being released back into space. The mechanism ensures that Earth's average temperature is kept between −18°C and 15°C. The gases which allow this phenomenon are not naturally highly concentrated in the atmosphere, but human activities have been increasing those concentrations. Therefore the atmosphere's characteristics and the greenhouse effect have been altered. This alteration is often considered to be the main cause of global warming.

Over the last century, carbon dioxide concentration in the atmosphere increased by 30%, and it is estimated that in 2011, CO_2, with approximately 459 million tonnes (Mt), represented 83% of the United Kingdom's greenhouse emissions (DECC, 2012) (Table 15.3). Moving to a low carbon economy is practical, achievable and desirable. It will require investment in new ways of generating energy, not a sacrifice in living standards.

Generally, the terms concrete and cement are used synonymously. In fact, although the production of cement affects the environment negatively (ie, through energy consumption and CO_2 emissions), compared to the other building materials, concrete is an eco-efficient material (Table 15.4).

The popularity of concrete makes it one of the most used materials and, therefore, one of the major contributors to CO_2 emissions. Because of this volume of production, the cement production industry is responsible for 5–8% of the overall worldwide CO_2 production (Scrivener and Kirkpatrick, 2008).

To respect the Kyoto Protocol, by 2012 the United Kingdom had to reduce its greenhouse emissions from their 1990 level by 12.5% (Concrete Centre, 2011). Moreover, the United Kingdom bound itself through the 2008 Climate Change Act to decrease its CO_2 emissions by 80% below their 1990 level by 2050. A midterm

Table 15.3 GHG emissions by gas type, 2011 (DECC, 2012)

	Greenhouse emissions (%)
Carbon dioxide	83
Methane	8
Nitrous oxide	6
Fluorinated gases	3
	100

Table 15.4 Embodied energy and embodied CO_2 for popular construction materials (Gartner, 2011; Hammond and Jones, 2011)

Material	Embodied energy (MJ/kg)	Embodied CO_2 (kg CO_2/kg)	Estimated consumption (Mt/year)
Concrete	0.88	0.123	23,000
Fired clay	3.00	0.230	4000
Glass	15.00	0.860	100
Steel	36.00	2.830	1400
Wood	10.00	0.230	3000

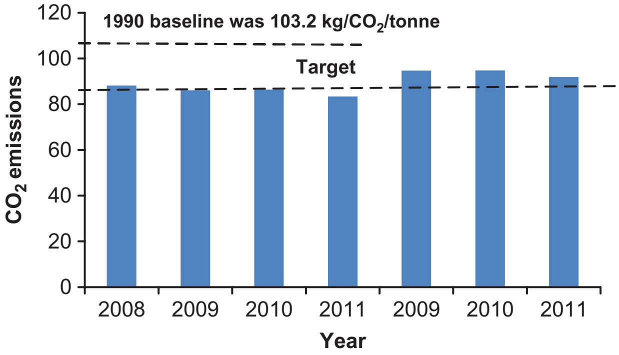

Fig. 15.1 CO_2 emissions for concrete mix production (Concrete Centre, 2011).

target has also been set to achieve a 34% reduction by 2020 (PAS, 2011). If the United Kingdom wants to reach this objective, all sectors must increase their efforts. From the document Concrete Industry, Sustainability Performance Report (5th report), it is clear that the concrete industry is still below its target, but a significant reduction has been achieved (Fig. 15.1). By using a standardised concrete mix, 4 years of collected data showed a 19% reduction of CO_2 compared to the 1990 baseline (Fig. 15.1).

15.2 Manufacturing of concrete

Concrete material is a mixture containing aggregate, cement, water, SCMs and/or minerals and chemical admixtures. Once hardened, the concrete will behave like solid rock materials with excellent mechanical properties and good resistance to chemical agents, frost cycles and fire.

15.2.1 Raw materials

15.2.1.1 Cement

Cement is a mixture of clinker and gypsum or calcium sulphate. Clinker is composed of various calcium silicates, including alite (C_3S); belite (C_2S); tricalcium aluminate (C_3A); calcium aluminoferrite (C_4AF) obtained by calcining limestone, clays and gypsum; and some SCMs, depending of the type of cement. The cement is the result of a complex production process involving three major steps (Fig. 15.2).

Step 1: Preparation of the raw material at the quarry

Cement is produced from a well-controlled mix of raw materials which are extracted from quarries located close to the cement factory to reduce the transportation distance and its related emissions. The raw material is mainly composed of limestone, marble or chalk as calcium carbonate $CaCO_3$ sources (about 80% of the mix). About 20% of secondary materials are used in order to adjust the chemical composition of the cement, such as iron ore, bauxite, shale, clay or sand (source of silica, alumina and iron oxide).

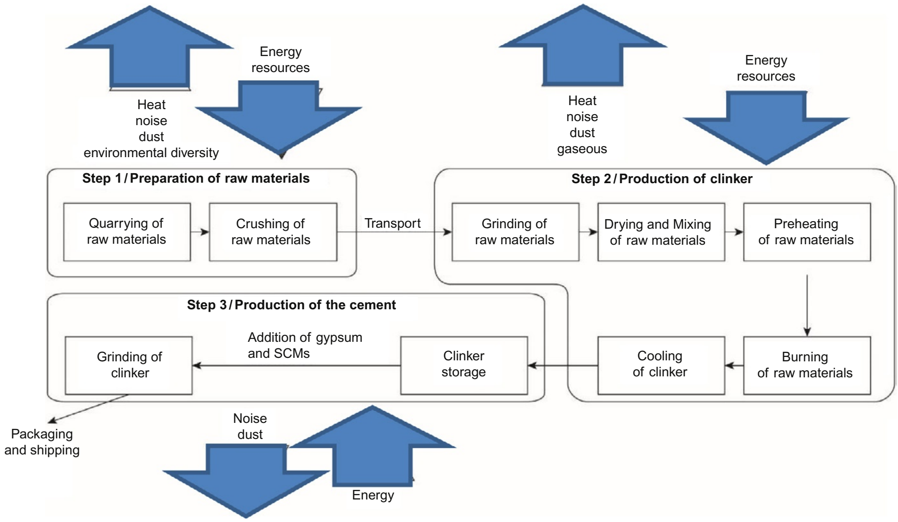

Fig. 15.2 Cement production and its major environmental impact.

Step 2: Production of the clinker

These raw materials, after being transported to the main factory, pass through a series of crushing, homogenisation and grinding processes in order to produce a fine powdered material (size about 50 μm). After these preparation stages, the process may change significantly from one cement plant to another, with an improvement of the efficiency of the plants to reduce energy consumption and gaseous emissions with a simultaneous increase in the productivity. In the original process, called wet process, the raw ground materials are fed into a kiln in the form of slurry at ambient temperature. This technique was first employed because it allows better control of the homogeneity of the raw materials and easier handling with less dust emission, resulting in more consistent cement properties. This approach requires, however, the use of a long rotary kiln (up to 225 m in length and 6 m in diameter, with a rotational speed of 1 to 4 rotations per minute) in order to increase the heat exchange surface. The major drawback of this technique is that two-thirds of the overall fuel consumption is necessary to evaporate the 40–45% of the water added in the slurry. Beginning in the early 1930s the process line was completed by adding a series of vertical cyclone preheaters, increasing the temperature of raw material up to 550°C before it enters the kiln, cutting by half the fuel consumption and reducing the length of the kiln (Allwood et al., 2011). This more complex system involves pre-calcination of the raw materials by introducing up to 60% of the total fuel directly into the preheater. During this process, 80–95% of the $CaCO_3$ can be calcined before it enters the kiln (Allwood et al., 2011). Finally, the temperature of the materials in the kiln is increased up to 1450°C in order to produce the so-called clinker. After leaving the kiln, the clinker is cooled down. Various techniques exist for this operation with their advantages and their drawbacks (Allwood et al., 2011). Note that a part of the hot air resulting in the cooling operation can be reused in the preheater.

Step 3: Producing the cement

The clinker is finally ground to a fineness of 300–1000 m^2/kg in combination with about 5% of calcium sulphate (gypsum or anhydrite). The calcium sulphate is essential for the control of the hydration reaction. In modern cement production, other raw materials can be ground with the clinker to produce blended cement. This aspect will be discussed in detail in Section 15.5.3. Finally, the cement is packed and transported.

15.2.1.2 Supplementary cementitious materials

Numerous supplementary powdered materials are used for concrete production, both for making blended cement and for the use as a mineral addition at the concrete factory. The use of these products is a matter of local availability, standards and industrial habits. It is estimated that, if the construction industry committed to using SCMs, CO_2 emissions of cement could be reduced by at least 5% and perhaps up to 20% (Worrell et al., 2001). SCMs can be divided in two categories according to the European standards (EN 206-1, 2014):

- Type I: Virtually inert addition (mostly minerals addition)
- Type II: Reactive pozzolana or hydraulic materials

These materials are used for two objectives: to reduce the clinker volume and therefore the cost and environmental impact of the concrete and to modify or improve concrete properties in the fresh and hardened states. The reactive materials (Type II) are composed of natural pozzolanas, mostly volcanic rocks, fly ash (FA), silica fume (SF), ground granulated blast-furnace slag (GGBFS), desulphurised (FGD) waste and purposely made materials such as metakaolin (Khatib and Hibbert, 2005; Khatib et al., 2015a,b,c; Mangat et al., 2006). Fly ash is a by-product of coal combustion, which is one of the cheapest ways to produce the power necessary to cover the needs of countries like China, India and the United States. Because of this fact, fly ash is expected to be available in large amounts up to 2050 (Malhotra, 2010). Fly ashes are composed of spherical particles, which can improve the flow properties of concrete by decreasing the inter-particle friction of the system (Sonebi, 2006). Moreover, it is shown that in general the use of fly ash will decrease the demand for water (average estimation of 3% for each 10% of cement replacement). Also, fly ash can improve cohesiveness and reduce both segregation and bleeding (Sonebi, 2002, 2004, 2006). However, it should be noted that fly ash will increase the setting time and reduce strength development over time, especially at the early-age stage. Therefore, using between 15% and 25% of fly ash as replacement of cement by mass is common practice in some applications, such as mass concrete, where early strength is not a concern and higher volume of up to 60% of the cement replacement in order to reduce the hydration heat.

Ground blast furnace slag (GGBFS) is a by-product of the steel industry, presented as glassy granular material formed when molten blast-furnace slag is rapidly chilled by immersion in water (American Concrete Institute, 2003). When used in concrete, GGBFS has a similar effect to FA: that is, it reduces hydration heat, improves durability in aggressive environments and reduces the permeability of concrete (Sonebi, 2005;

Sonebi et al., 2008). GGBFS can be used with an optimum replacement rate of about 50%, but in some applications, up to 70% or 80% can be used (Sonebi, 2005).

SF is another industrial by-product used to improve the properties of concrete, including rheology (Sonebi, 2010), strength development and durability. When adequately used, SF will indeed increase the strength and durability of concrete by reducing its permeability. Furthermore, when used at low replacement dosages (up to 3%), SF can improve the rheology of concrete (Sonebi, 2010).

Metakaolin is produced by burning kaolin at high temperatures of about 800°C, and then it becomes metakaolin that can be used as SMCs. The main benefit of this material is that it greatly increases the concrete's strength (Khatib, 2008; Khatib and Hibbert, 2005; Sonebi et al., 2013a; Wild et al., 1996). Nevertheless, metakaolin is neither a waste nor a by-product of industry, so it requires proper consumption of energy. But the heat temperature can be higher because of the large surface area (Sonebi et al., 2013b). Moreover, unlike clinker, the reaction leading to metakaolin does not release CO_2 (Potgieter, 2012).

Another type of SCM still in the development phase is rice husk. It has an interesting future as a cementitious material in concrete. Rice husk ash (RHA) is produced by burning the residual husk from the production of rice. The estimated volume of RHA is about 25 Mt per year (Malhotra, 2010). Burned at an adequate temperature, ash containing a high amount of SiO_2 is produced and can be used successfully as replacement for cement. Some developments in burning techniques can produce highly reactive RHA for the design of HPC (Nehdi et al., 2003).

The Type I inert fillers used for concrete production are mainly finely ground rock powder from various mineralogical origins. The most used ones are limestone fillers, which is (an on-purpose produced is it not waste) material and is presented as a fine powder. Limestone powder can improve the properties of concrete by increasing the packing density of the solid matrix, as the filler effect. This filler effect will influence both fresh and hardened properties of concrete (Svermova et al., 2003). At fresh state, the use of mineral fillers will improve flow properties, which explains the great popularity of this material for the production of self-consolidating concrete. At hardened state, the filler can increase compressive strength and durability properties when used at an optimum replacement rate. If large amounts of inert filler are produced, industries, such as the marble- and granite-sawing industries, will also produce by-product filler (Yahia et al., 2015).

15.2.1.3 Aggregates

Aggregates are coarse, solid particles up to a few tenths of a millimetre in size which are used to produce durable concrete. A large variety of aggregates are used for concrete production. The ones which are mostly used for conventional concrete are sand and gravel composed of crushed quarry or riverbed washed rock particles. Some artificial aggregates are also used. These include light aggregates made from geosynthetic materials (or reduce the weight and improve sonic insulation, or industrial by-products like slag, or waste products like rubber tires) (Turatsinze et al., 2007).

Recycled aggregates used in concrete are popular because of the volume of demolished concrete generated from existing buildings, the low availability of natural aggregates, and the necessity to reduce the environmental impact of new concrete structures. Recycled aggregates have two origins: washed aggregates from fresh concretes (gathered in concrete plants and delivery trucks) and, especially, aggregates from old, demolished concrete. Recycled aggregates from old concrete are obtained by crushing the old concrete and, therefore, comprise the original natural aggregates and a residual layer of mortar. This peculiarity of recycled aggregates leads to special properties that need to be taken into account for the design of new concrete and will limit the volume of possible replacement in order not to affect concrete properties. Recycled aggregates usually present a low density, very high water absorption, a low resistance to abrasion and, depending on the origin, the possibility of minor amounts of secondary materials, such as asphalt, wood, ceramic and steel (González-Fonteboa and Martínez-Abella, 2008). Because of the great variability of the recycled aggregates, it is difficult to define a replacement rate, which can vary from 20% up to 50% as it will influence fresh and hardened properties.

15.2.1.4 Chemical admixture

Chemicals are becoming essential for the production of modern concrete, especially with the development of HPC (Sonebi and Khayat, 2001), including SCC (Sonebi, 2004; Sonebi and Bartos, 1999, 2002; Zhu et al., 2004), ultra-high-performance concrete (UHPC), and so on. They are based on chemical organic or inorganic molecules and are used to improve various properties of concrete, such as flow properties (water reducer agent, WRA, and high-range water-reducer agent, HRWRA), stability (viscosity modifying agent, VMA) (Sonebi, 2002, 2006; Sonebi et al., 2013a,b), air content (air entraining agent, AEA), shrinkage (admixture reducing agent), for example. WRA and HRWRA are the most used chemical admixtures with 40% and 38%, respectively, of the total amount of admixtures sold in Europe (EFCA, 2006). These admixtures play a major role for the concrete industry as they can be used for main objectives. The first objective is to reduce the water content of concrete to improve strength and durability while maintaining adequate flow properties. The second objective is to increase flow properties of concrete while maintaining the same water content (Sonebi, 2009). These admixtures are at the origin of the development of high-performance, self-consolidating concrete (Sonebi and Bartos, 1999, 2002). The main types are sulphonated naphthalene formaldehyde, sulphonated melamine formaldehyde, vinyl copolymers and polycarboxylic ether-based. They have an eco-profile close to that of cement with a CO_2 emission of 0.72 kg CO_2/kg compared to 0.842 for cement (not clear), NO_x of 1.8 g/kg for 3.65 g/kg, but a rather higher SO_x (not clear) with 3.6 for 0.53 g/kg (Van Den Heede and De Belie, 2012). However, these admixtures have a negligible effect on the overall concrete environmental impact because such a small amount is used in concrete, amounting to only a small percent of the cement mass.

15.2.1.5 Water

Water is essential in the entire process of producing concrete. First, water's consumption in the process averages between 300 and 400 L/tonne of cement (wet method). Secondly, it is used for the design of concrete to ensure chemical hydration of the cement with an average value of 300 L/m^3 of concrete. Finally, large amounts are used for washing all the production devices and transportation trucks. Moreover, in order to reduce the contamination by organic substances, chloride and alkali, most concrete production lines use drinkable water. In order to reduce the environmental impact of the industry, most concrete plants are now equipped with water recycling devices for the reuse of the washing water (European Concrete Platform and Asbl, 2009).

15.3 Life cycle aspects and environmental impact of concrete

15.3.1 Concrete's carbon footprint definition

Regarding the cycle of concrete production as a construction material—that is, from cradle-to-gate—(Table 15.5) 74% to 81% of the total CO_2 emission is attributed to the manufacturing of cement (Flower and Sanjayan, 2007). The second major impact is the GGBFS, and third one is coarse aggregate.

The cement carbon footprints are issued from two major sources, the decarbonation of lime during the burning process and emissions resulting from the production of heat necessary to increase the temperature up to 1500°C (Fig. 15.3). Some minor emissions can also be attributed to transportation, grinding of raw materials and clinker. The carbon footprint of cement has an impact on a global, regional and local scale (Huntzinger and Eatmon, 2009) and can be attributed essentially to carbon dioxide (CO_2), nitrogen oxides (NO_x), Sulphur oxides (SO_x) and dust emissions.

On a local scale of cement production, the major impacts are noise and air quality (dust emissions) from mining and crushing operations of raw materials and during the cooling process of clinker. The mining process will also lead to natural disturbances (landscape and ecosystem changes). On a regional scale, cement production will have two major impacts, acidification of rain and eutrophication (excess of nutriments) in an aquatic environment and agricultural soils. These phenomena are due to the emission of nitrogen oxides (NO_x) and sulphur oxides (SO_x). These components are due

Table 15.5 CO_2 emission factor for concrete production (Flower and Sanjayan, 2007)

Activity	Emission factor	Unit
Coarse aggregate (Granite)	0.0459	tonne CO_2-e/tonne
Fine aggregate	0.0139	tonne CO_2-e/tonne
Cement	0.8200	tonne CO_2-e/tonne
Fly ash (F-type)	0.0270	tonne CO_2-e/tonne
GGBFS	0.1430	tonne CO_2-e/tonne
Concrete batching	0.0033	tonne CO_2-e/m^3
Concrete transport	0.0094	tonne CO_2-e/m^3
Onsite placement activities	0.0090	tonne CO_2-e/m^3

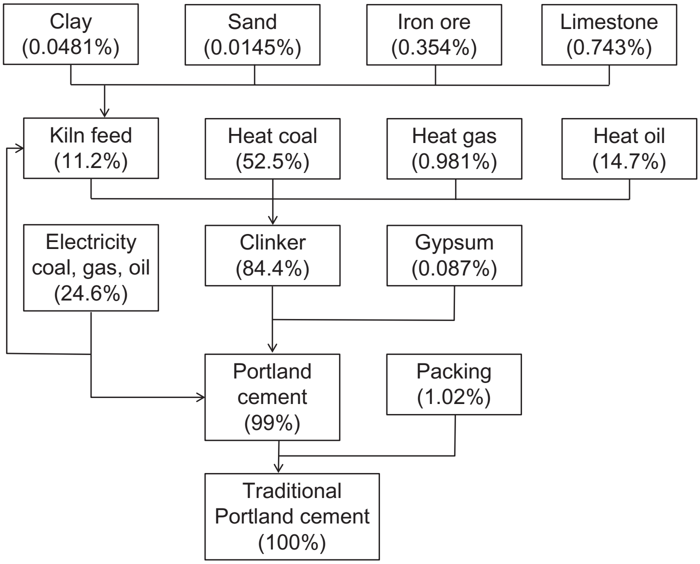

Fig. 15.3 Life-cycle analysis (overall environmental impact) of traditional Portland (cradle-to-gate) of traditional cement production (Huntzinger and Eatmon, 2009).

mainly to the combustion process of fossil fuels and the use of vehicular transportation (Josa et al., 2004). Finally, on a global scale, the CO_2 emissions are due to two sources that play major roles. The first and main source is the decarbonation of lime (responsible for 60–65% of the CO_2 emissions). This is a necessary process to produce clinker following the chemical reaction given in Eq. (15.1).

$$3CaCO_3 + SiO_2 \rightarrow Ca_3SiO_5 + 3CO_2 \tag{15.1}$$

The second source is the CO_2 produced by the combustion of fossil and alternative fuels used in the burning process (responsible for 35–40% of the CO_2 emissions).

15.3.2 Assessment of carbon footprint of concrete

The ecological footprint of a product assesses its impact on the environment. The ecological footprint is commonly defined as the area of productive land and sea needed to create the product and then make it disappear. A carbon footprint can also be assessed quantitatively. In this case, it refers to the GHGs emitted by a product from the time of its creation to the end of its life. As discussed earlier, CO_2 is responsible for most emissions, which is why the carbon footprint is generally expressed in kilograms of CO_2.

A tool is used to determine what improvements need to be made and where to make them to reduce the emissions. Indeed, the concrete industry will be able to spot the weaknesses of the supply chain and, therefore, gather the funds to find innovative technologies to lessen these emissions. Moreover, it is important to know that carbon releases can be classified as follows (Harmouche et al., 2012):

- *Direct footprint*: This refers to the emissions released during manufacturing and transportation
- *Indirect footprint*: This refers to emissions for the entire life cycle, from manufacturing until the end (recycling or landfill)
- *Embodied carbon:* This corresponds to the same CO_2 emissions as the direct footprint does
- *Operational carbon*: This corresponds to CO_2 emitted by a product during its use

The difficulty of assessing the carbon footprint of concrete is due to the numerous parameters that must be taken into account. Indeed, as discussed in a previous Section 15.2.1, different materials can be used in concrete production, including different cement types, SCMs (type, replacement content, etc.), admixtures (type, dosages, etc.), aggregates (type, size of particles, shape and grading, etc.). Furthermore, each of these materials will have a different influence on concrete properties with great variability in strength and durability. On the other hand, the concrete design itself can greatly influence its properties (water-to-cement ratio, cement volume per m^3 of concrete, aggregate size and volume, etc.). The use of special concrete, such as UHPC and fluid and lightweight concretes, can also introduce another variable.

Moreover, the chosen boundaries will strongly affect the final evaluation of a carbon footprint for concrete. A solution (cement type) that seems better at production (lower environmental impact) cannot be so effective when used at concrete scale or event at building scale. When doing a life-cycle analysis (LCA, also called a cradle-to-grave analysis) of concrete, one major difficulty is defining the functional unit (ISO 14044, 2006), because the tiniest modification in concrete properties can lead to big differences in the construction process, thereby changing the chosen functional unit (Van Den Heede and De Belie, 2012). It is, therefore, necessary to include the factors of strength and durability/service life when defining a functional unit.

The LCA and environmental impact of concrete from cradle-to-grave should be performed from the time the raw materials are extracted, throughout the manufacturing process, throughout the lifetime of the building and up to the time it is recycled (Fig. 15.4).

The ecological footprint of concrete is important because it requires lots of water, raw materials (aggregates), cement and the energy to produce the concrete. The production of concrete consumes numerous natural resources. Indeed, the construction industry consumes 40% of the raw stones, sand and gravel used worldwide and 16% of the water (Harmouche et al., 2012). At the end, concrete debris must be dealt with. For example, in the United Kingdom, the debris used to be put in landfills, but that is now forbidden.

Even if concrete does contain only 15% cement by unit volume, it remains primarily responsible for concrete's carbon footprint (CCF). Indeed, each tonne of cement produces about 1 tonne of CO_2 (Concrete Centre, 2011). It is worth knowing that the cement industry represents between 5% and 7% of the world's CO_2 emissions (Meyer, 2009). In the United Kingdom, concrete manufacturing and delivery is responsible for more than 2.5% of its total CO_2 emissions (Concrete Centre, 2011).

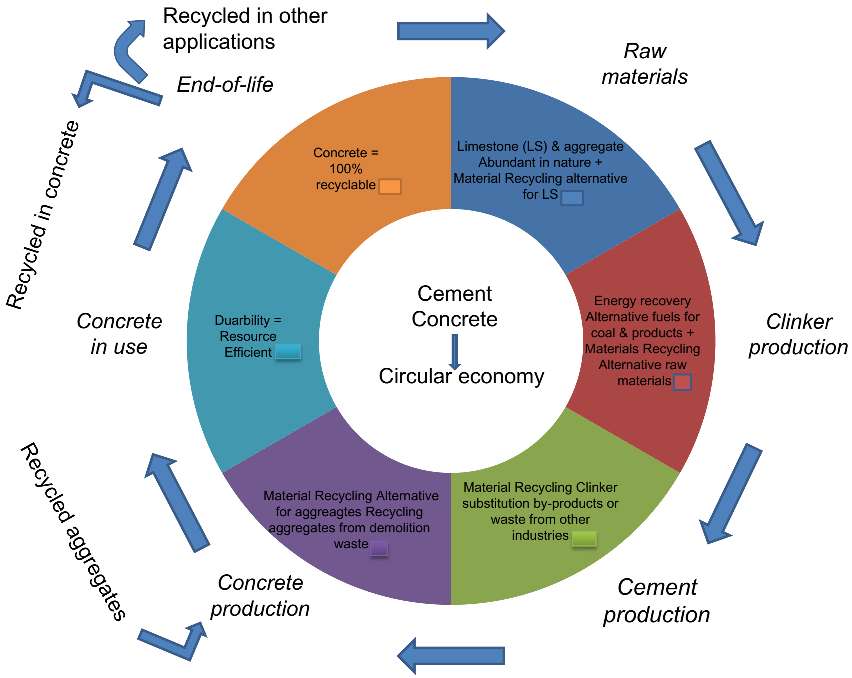

Fig. 15.4 Life cycle of concrete from cradle-to-grave (CEMBUREAU, 2014).

Table 15.6 **Estimation of the CCF (Concrete Centre, 2011)**

Element	CO$_2$ emissions (kg/t)
Average concrete	125
Dense concrete aggregate block	75
Aerated concrete block	240

Nevertheless, it is really complex to determine the CCF. Some preliminary figures have been reported by Concrete Centre (2011), as shown in Table 15.6.

15.3.3 Life-cycle assessment of concrete

About 40% of the end use of energy is attributed to the building sector, which is even more than for the transportation and industry sectors (European Concrete Platform and Asbl, 2009). In the case of buildings, the increase in the demand for new structures is 67% for residential buildings, and the service building area is expected to triple by 2050 (International Energy Agency, 2010). This increase in the demand for structures needed to meet the needs of the world's growing population cannot be covered without the use of an increasing amount of concrete. In regard to concrete structures, it has been shown that concrete production has an important environmental impact.

But, looking at a LCA of a concrete structure, the in-service phase and the end-of-life phase can be taken into account when designing the entire project. In this case, the properties of concrete could be used advantageously in order to produce a zero-impact building.

LCA is a major concern when planning any construction project because it will strongly influence choosing the right materials for the right application. Using a LCA not only helps to make decisions about materials (eg, concrete versus steel) but also can improve the use of a material (ie, concrete) by adjusting its type and properties for specific applications.

15.4 A concrete structure, from cradle-to-grave

15.4.1 Concrete production

As seen previously, the major impact of concrete production comes from cement manufacturing. The other activities also have some minor impacts. The second major source of CO_2 in the production of concrete is attributed to the production of aggregates. In terms of impact, the most important aspect of aggregate production is not the extraction or production processes, but transportation, which makes it a necessity to use locally available aggregate. The lack of space and the difficulty of creating new quarries or increasing the size of existing quarries make it difficult to produce aggregates with a low impact. The other environmental aspect of aggregate production is its effect on biodiversity at extraction sites.

In the mixing plant, most CO_2 comes from the mixing process powered by electricity consisting of 96% of the used energy (Flower and Sanjayan, 2007), and a small amount comes from the fuel used by the various lifting equipment. The impact of transporting concrete can be limited by the strategic placement of concrete production plants near major cities and the production of concrete at construction sites, if necessary.

The placement of concrete requires some energy to fuel devices for craning, pumping vibrating and finishing the concrete. Use of energy at the final stage can be reduced by utilising modern concrete such as SCC (Kwasny et al., 2015; Sonebi, 2004; Sonebi and Bartos, 1999, 2002) in order to facilitate the concrete operations and to eliminate the need for vibration to consolidate concrete.

During the production and placement of concrete, a large amount of water is used in order to wash the devices and fresh concrete waste is consumed. In order to reduce the impact of concrete production, both washing water and aggregates (after washing) from fresh concretes are reused in the concrete production line.

15.4.2 Operational phases

The operating phase of a building is defined as the period in which the building can withstand the conditions of normal use. For buildings, this period lasts at least 50 years and can last several hundred years. Because of this long service life, the operational phase of a building is the phase with the most environment impact.

The operating phase of buildings contributes 10% of global CO_2 emissions, without taking into account the indirect emissions because of the electricity produced as a result of the high consumption of energy for heating and cooling systems, water heating, lighting and use of consumer products and business equipment (International Energy Agency, 2010). This is where concrete can play a major role because of its high thermal mass which can be used to store heat produced by the sun during the day and release it at night, thus reducing temperature fluctuations during the day. This effect can reduce the amount of energy consumed by heating (by 2–15%) and cooling devices (by 50%) (European Concrete Platform and Asbl, 2009).

15.4.3 End of life

The end of life of concrete is defined as when the structure can no longer play its role in ensuring the safety of users. The end of life of a concrete structure is directly related to the durability of the materials used and the external constraints, such as mechanical charges and exposure conditions (frost, deicing salts and aggressive chemicals, etc.). In some cases, where the structure is prematurely damaged or is difficult to replace, repairing operations can be used to extend a structure's service life. These operations are often of concern from both financial and environmental aspects.

15.4.4 Demolition and recycling

In the context of modern building practices, concrete structures are no longer demolished but are deconstructed. Deconstruction differs from demolishing by its selective dismantlement of a building's various materials for optimal reuse and recycling. In the case of concrete, the solid material is crushed in order to produce recycled aggregates, which can be used as aggregate for new concrete structures.

15.5 Lowering the environmental impact of concrete

In Europe, cement production decreased by 26.9% from 1990 to 2012, whereas CO_2 emissions decreased by 38.6%, showing an improvement in the cement production (CEMBUREAU, 2014). However, to reach the objectives of various sustainability programs, further efforts must be made in order to improve every step in the concrete production line. For concrete, the main solutions for reducing the environmental impact of modern construction are (Flatt et al., 2012):

1. Partial cement (clinker) replacement with SCMs
2. Development of alternative binders
3. Broader use of concrete mix designs that limit cement content
4. Recycling of demolished concrete in new concretes
5. Enhancement of durability (designing new infrastructures for longer service life)
6. Rehabilitation of existing infrastructures (extending the service life of existing infrastructures) rather than demolishing and reconstructing

Some other technical solutions to improve cement production are also of interest, including CO_2 capture and storage. If this solution were to present a major impact on the final objective, it would need technological breakthroughs that are still in the research phase and are not planned to be ready for users before 2030, but they could potentially capture up to 45% of the CO_2 produced by cement (IEA and WBCSD, 2010). By 2050 the baseline emissions will be 2.34 gigatonne (Gt) but will be reduced to blue emissions of 1.55 Gt by the contribution of energy efficiency (10%), use of alternative fuels and other fuel switching (24%), clinker substitution (10%) and carbon capture and storage (56%) (IEA and WBCSD, 2010).

15.5.1 Decreasing the content of clinker

The worldwide use of blended cement in the production of cement is already a commonly used improvement, with significant investments for research made by cement producers (Table 15.7). Many blended cement types exist because of the great variability of usable SCMs. These cement types are produced by replacing part of clinker with SCMs, which leads to cement with unchanged or improved properties for both general and special applications. The same properties must be maintained; otherwise, for a functional unit (concrete beam) and on a product scale (cement), more cement may be necessary to obtain the same durability. Therefore the beneficial impact of replacing a part of clinker can be useless (Li et al., 2004). In the European context, binary cement incorporates up to 35% of each type of SCM even in the case of inert mineral fillers, and ternary cement uses up to 80% in mass replacement of clinker (EN 197-1, 2001). A final trend is the development of quaternary binder, but a lot of improvements remain necessary in order to ensure a better understanding of the interactions between clinker and SCMs.

Table 15.7 Cement types produced by Holcim 1995–2009 (Schneider et al., 2011)

	Production by Holcim (%)		
Year	**1995**	**2000**	**2009**
OPC	56	45	20
Slag cement	8	10	9
Pozzolan cement	9	8	9
Fly ash	4	5	26
Limestone powder	7	13	14
Multiple blend cement	11	12	17
MIC and other cement materials	2	2	2
Masonry cement	<1	<1	0
Oil well cement	0	<1	0
White cement	2	2	2
Special binder	1	3	0

Table 15.8 Reduction in CO_2 emissions through use of blended cements (WBCSD, 2013)

Year	1990	2000	2005	2006	2007	2008	2009	2010	2011	2012	2013
kg CO_2/tonne cementitious	736	720	666	659	651	652	643	642	613	618	610

As shown, the use of blended cement reduced CO_2 emissions by about 20% from 1990 to 2013, and a reduction in the cement-to-clinker ratio to 71% is expected in 2050 (Table 15.8).

15.5.2 Improving cement production

As the CO_2 resulting from the decarbonation of limestone during the calcination process is a fixed amount by clinker volume, the two major solutions are optimisation of the heating process to reduce the energy needed to reach 1450°C and the use of blended cements.

The improvement of a specific burning process leads to no or very little change in the reduction of CO_2 emissions, but a change from a wet process to a dry process with preheating and precalciner kilns can lead to significant improvements (WBCSD, 2013). The energy used by a wet kiln is estimated to be between 5.9 and 6.7 GJ/tonne of clinker, whereas a kiln with preheating and precalciner kilns uses only 2.9 to 3.3 GJ/tonne of clinker (reduction by 50%). Shifting the cement production from wet to dry with preheating and precalciner kilns can lead to a reduction of 20% (International Energy Agency, 2010) of the energy needed and 17% in the amount of CO_2 emitted per tonne of clinker (Damtoft et al., 2008). It can be seen that the increase in part of the clinker produced by this technology could partially overcome the increase in cement production (WBCSD, 2013). These savings could be increased by the use of more alternative fuels (Nielsen and Glavind, 2007) (to coal, which actually accounts for 60% of the fuel used in cement production).

15.5.3 Use of SCMs

The use of SCMs in concrete is different from using them to produce blended cements. There is no guarantee on the potential strength achieved by a mixture of cement and SCM because of the great variability in the physicochemical properties of SCM. In order to control the amount used in concrete, use of SCM has been standardised through the *k*-value concept (Smith, 1967). This concept is based on the potential reactivity of each SCM, which helps in fixing a maximal replacement rate of cement to achieve the same mechanical and durability properties. The *k*-value for each addition differs depending on the type, on the concrete exposure conditions (frost, salt, sulphate, etc.) and on the local national standards. Depending on the type of SCM, the volume used and the targeted concrete strength, the savings in terms of CO_2 emissions can be more than 20% (Table 15.9).

Table 15.9 **CO_2 emissions generated by commercially available concretes (Flower and Sanjayan, 2007)**

	100% GP cement		25% Fly ash		40% GGBFS	
Strength (MPa)	25	32	25	32	25	32
Emissions (tonne CO_2-e/m^3)	0.290	0.322	0.253	0.273	0.225	0.251
% CO_2 reduction	0	0	13	15	22	22

15.5.4 Other alternatives

The previous solutions for improving the environmental aspect of concrete are widely used and remain of interest for further development, and some new, challenging solutions are also being studied but will need either breakthrough technology or huge investments in development. So far emissions of CO_2 are inherent to cement production, so finding ways to prevent this gas from getting into the atmosphere need to be explored, as illustrated next.

- *Carbon capture and sequestration (CCS)*: This process consists basically of capturing the CO_2 before it is released into the atmosphere and then compress it and store it underground (in mines, caves, oceans). Unfortunately, currently this process remains quite expensive and energy-consuming. Nevertheless, it is a new technology which can be improved in the future. On the other hand, it can also be argued that far from dealing with the problem, CCS is just a way to avoid the problem and leave it for later. Therefore it is totally in contradiction with sustainable development, which is defined by the Brundtland report (United Nations, 1987) as a 'development that meets the needs of the present without compromising the ability of future generations to meet their own needs'.
- *Biofuel production*: Although this approach is not yet totally explored, some researches have shown that captured carbon can be used in the production of algae that can then be transformed to biofuel, agricultural fertilizer or even animal protein (Potgieter, 2012).
- *Electrochemical carbon reduction (ECR)*: This process involves creating a reaction which will transform carbon into formic acid. This product is often used in the pharmaceutical industry. It is usually manufactured with high energy consumption but ERC requires less energy (Potgieter, 2012).

The first approach will be to reduce the volume of CO_2 in the production of cement which is emitted into the atmosphere. But the most interesting solution in the short-term will be the development of alternative binders with a lower environmental impact, and some proposed solutions already provide excellent results on a laboratory scale or event in pilot projects (Aldred and Day, 2012; Duxson and Provis, 2008; Owens et al., 2010). Alkali-activated cements are aluminosilicate–pozzolana-based materials (glass furnace slag, natural volcanic glass, manmade glass, fly ash, metakaolin). The activation occurs between the alumina-rich pozzolana and a strong alkali base, which dissolves the silicate and aluminate groups to form a cementitious gel to form the structure of the matrix. The so-called geopolymer is proven to have similar mechanical properties as cement-based concrete with a lower environmental impact (Duxson et al., 2007). This solution is already implemented in Australia by using fly ash and slag (Duxson and Provis, 2008). This solution is, however, hindered by some practical aspects, including the robustness of the design of the mix, the heat needed for a proper reaction to occur in a reasonable amount of time, the limited amount of available aluminosilicate compounds, the handling of the alkaline activator on the job site, the cost, and the environmental impact of the activator (Flatt et al., 2012).

Belite-rich cement, with a combination of calcium sulphoaluminate and calcium sulphoferrite, have been tested with success. Compared to ordinary Portland cement, belite-rich cement contains relatively higher belite (C_2S) and lower alite (C_3S) contents (Chatterjee, 1996). Belite-rich cement produces performances comparable to cement with the production of 20–30% less CO_2 (Li et al., 2007).

15.5.5 Uses for modern concrete

When designing sustainable concrete structures, two main approaches are used in order to reduce the environmental impact of the materials:

- Development and use of eco-products, that is, products that have a lower impact at the product level, while maintaining overall mechanical properties
- Development and use of eco-efficient products which can have a high impact at the product level but a lower impact during a structure's lifetime by improving mechanical and durability properties

Some specially designed concretes have been developed with the eco-efficiency of products at a structural level in mind. This is particularly the case of HPC, including SCC (Hossain et al., 2013; Sonebi and Bartos, 1999, 2002). First, SCC will not act directly on the environmental aspect of sustainability, but rather on economical and societal aspects. When properly designed from the beginning, a construction made of SCC brings economic benefits by increasing the productivity because of the higher casting rate and the reduction in manpower which results in the elimination of vibration. This latter aspect also has a major societal aspect because it reduces the noise at construction sites and concrete factories and removes the risk of injury to workers related to crowded construction sites and vibration (Nielsen, 2007). The use of SCC in concrete factories (Yahia et al., 2011) can also be a solution for increasing mould service life and saving energy. In the case of concrete pipes, this reduction can be about 1.0 GWh (De Schutter et al., 2010). Note also the reduced need for post-treatment of a surface by plaster, which accounts for 0.57 kg of equivalent CO_2 per m^2, or by paint because of the better finish of concrete surfaces (Witkowski, 2015). This latter aspect can also increase the CO_2 uptake by concrete carbonation because the concrete surface has no applied post-treatment. The major issue of SCC is that proportioning of a mix involves a significant amount of cementitious material in order to improve the workability of the concrete. This problem can be overcome with the development of high-performance chemical admixtures, use of high SCM content (Diederich et al., 2013), and an adequate selection of the granular skeleton of solid particles to achieve proper rheological properties of concrete. The latter approach has allowed the production of Eco-SCC with 40% fewer CO_2 emissions than standard vibrated concrete, along with a 150-year service life (Mansour et al., 2013).

The development of high-strength concrete and high-strength, fibre-reinforced concrete is also a new approach to sustainability in construction materials. These concrete types are designed with a relatively greater amount of binder (cement and SCM) and chemical admixture, which achieves better performance than conventional concrete, including 3–16 times more compressive strength, 10 times more flexural strength and 10–100 times more durability than conventional concrete does (Wang et al., 2015). These high performances allow a reduction in the size of elements for similar structural performance, therefore leading to a lower volume of concrete for the same structure. This reduction in volume can lead to a reduction of 65% in raw materials consumed, 51% in the primary energy used, and 47% in CO_2 emissions (Batoz and Rivallain, 2009).

15.5.6 Recycled aggregates

Currently in the United Kingdom, concrete debris is not sent to landfills but is treated so it can have a second life in the construction sector. Indeed, concrete is a material that can be fully recycled. In 2011 recycled and secondary aggregates represented about 5.3% of all aggregates used in concrete (Concrete Centre, 2011). This material is used mainly in road construction, but recycled concrete aggregates (RCA) can count for up to 30% of the aggregates in a concrete mix. In the United Kingdom recycled and secondary materials represent 28% of the total aggregates used in the marketplace, the highest in the Europe. The precast concrete industry provides greater opportunities for using recycled aggregates over to 20% (Concrete Centre, 2011). However, the following issues need to be considered when using RCA:

- ✓ It might contain contaminants that can alter the strength and durability of concrete
- ✓ Concrete's strength will decrease by up to 40%, and its permeability will increase along with creep and shrinkage deformations (depending upon the amount of recycled materials used)

15.6 Case studies

15.6.1 CO_2 uptake

CO_2 uptake by concrete is a natural process related to the carbonation process during which calcium hydroxide ($Ca(OH)_2$) from the concrete will react with carbon dioxide (CO_2) to form calcium carbonate ($CaCO_3$). During the service life of a building, this phenomenon must be limited because it will lower the pH of the concrete leading to a higher risk of corrosion of a steel reinforcement. In fact, in a concrete structure, carbonation is a slow reaction which depends on the porosity of the concrete, which will govern the rate at which the CO_2 penetrates and reaches the concrete which is already behind the carbonated layer. During the service life of a structure, this phenomenon is rather difficult to estimate because of the numerous parameters involved, such as the exposed surface of the concrete, the permeability of the concrete, the humidity of the concrete, the use of SCM, the presence of surface treatment and so on. Research by the Swedish Cement and Concrete Research

Table 15.10 CO_2 balance of concrete in the various Nordic countries, based on total cement production (Kjellsen et al., 2005)

	Denmark	Norway	Sweden	Iceland
	Year 2003			
Total CO_2 emission (million metric tonnes)	58	43.2	57.6	2.2
Total CO_2 emission from cement production[a] (million metric tonnes)	2.5	1.4	1.9	0.06
CO_2 emission from calcination process of cement production[a] (million metric tonnes)	1.3	0.9	1.1	0.028
Calculated CO_2 uptake (carbonation) after 100 years (million metric tonnes)	0.34	0.22	0.24	0.021
Percent of CO_2 taken up by carbonation compared of the CO_2 emitted in the calcination process	26	24	22	75

[a] Total cement and clinker production.

Institute showed the value of CO_2 uptake that can be expected during the service life of a building (Kjellsen et al., 2005) (Table 15.10).

After demolition of a structure, the crushed, recycled aggregate gathered from concrete can be subjected to carbonation. While the exchange surface between aggregates and air is greater because of the crushing, the carbonation will be higher. It was found that 60–80% of CO_2 released during calcination is reabsorbed by concrete mixtures with *w*/*c* of 0.6 or higher for the grain size of 1–8 mm (Lagerblad, 2005). Moreover, it has also been shown that the use of recycled aggregates that have been carbonated will increase density and lower water absorption, leading to better concrete properties, including better flow and compressive strength (Zhang et al., 2015).

15.7 Future trends

Through significant research and development (R&D), the construction industry constantly pursues improvements regarding materials and operation techniques that will improve sustainability. As with other industries, it took time for the construction industry's innovations to spread from R&D to the field. Since the modern development of the concrete industry, major improvements have been made in order to improve the mechanical properties of materials and to produce impressive and environmentally friendly buildings. The need of a sustainable development of construction industry has already made sense to all the actors; however, the necessity to improve not only the material properties but also the whole process starting from the design of a construction on to the end of its lifetime of use is a rather long process.

The construction process will be done through a performance-approach instead of a classic approach. Indeed, once a GHG life-cycle assessment is done, it can be used to point out the most polluting stage of the concrete's life so that improvements in that regard can be made. A life-cycle assessment is, so far, the most complete framework for estimating the carbon footprint of a product. It does remain complex and must be popularized across the industry. Moreover, some practical questions need to be dealt with to make sure that when it comes to calculations, everyone follows the same rules and collects proper data.

In practice, it is possible to establish that, when used for the construction of a structure, concrete will prove to be quite unsustainable during the construction phase but advantageous for the operational phase and demolition. Therefore concrete's thermal mass must definitely be included in any structural design.

The CO_2 emissions, taken into account for the CCF, come mainly from the heating of raw material to produce clinker. To reduce those GHG releases, several options can be studied: for example, clinker replacement, alternative fuel, improved kiln, carbon capture and recycled material. However, all the methods are not equally implemented because of trust issues, cost and sometimes even ignorance of their existence.

The use of SCMs remains the simplest technique to reduce concrete's CO_2 emissions, and it has already spread throughout the cement industry; however, its benefits can still be enhanced by a wider application. The only obstacle to this is the insufficiency of early strength with SCMs. Therefore, research must be carried out to improve their efficiency, and the construction sector must be ready to make some sacrifices for a sustainable development. Other SCMs, such as glass powder, can be used (Idir et al., 2009). The use of recycled and secondary aggregates can contribute to reducing the carbon footprint and preserving natural resources.

Furthermore, it is necessary to carry out more research about concrete's absorption of CO_2, mainly to establish a real estimate of its benefits but also to assess to what extent it affects the RCA quality.

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Durability of sustainable construction materials

16

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16.1 Introduction

In spite of the improved understanding of many common causes and consequences, the failure of materials is still of great concern to the building and civil engineering construction industry. Freezing and thawing in the temperate regions with repeated freeze–thaw cycles can cause severe deterioration of concrete. This freeze–thaw process induces large stresses in concrete leading ultimately to damage. The damage is accelerated, particularly in pavements by the use of deicing salts, often resulting in severe scaling at the surface. Sulphates in soil and groundwater are resisted by using suitable cementitious materials and a properly proportioned concrete mixture subjected to adequate quality control. Other physical degradations such as abrasion and cracking can cause concrete surfaces to wear away.

Recent years have seen changes in environment legislation related to the disposal of industrial waste materials. These changes have forced growing interest in the use of a wide range of supplementary cementitious materials (SCMs) and recycled materials, for example, pulverised-fuel ash (PFA), ground granulated blast-furnace slag (GGBS), silica fume (SF), flue gas desulphurisation (FGD), metakaolin (MK), and waste-paper sludge ash (WSA), to partially replace Portland cement (PC) in concrete, and recycled concrete as an alternative to aggregates. An advantage of such utilisation of SCMs is the contribution toward the conservation of natural resources.

This chapter reviews recent research activities on the durability of concrete and examines the nature of degradation of materials and the effects of SCMs on concrete performance. Also, current specifications for durable structure are discussed. This chapter outlines durability of sustainable construction materials. It examines the important causes of material deterioration and discusses how to prevent such damage. The chapter begins by discussing the nature of degradations of materials and addressing the reasons for durability problems. The chapter identifies the effect of using sustainable construction materials on durability and examines the processes involved in building defects and failures.

16.2 Degradation of materials

Durability of concrete is defined as its ability to resist deterioration processes that may occur as a result of weathering action caused by the interaction with its external environment (Alexander et al., 2007; Cai and Liu, 1998; Pade and Guimaraes, 2007;

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Song et al., 2008; Yoon et al., 2007) or reactions that may occur between the constituent materials including the internal contaminants present (Fernandes et al., 2004; Kockal and Turker, 2007; Owsiak, 2004). The American Concrete Institute (ACI) defines durability as 'the ability of a material to resist weathering action, chemical attack, abrasion, and other conditions of service' (ACI CT-13, 2013). The deterioration is largely the result of physical events (cracking, frost, abrasion, and fire) (Felicetti, 2006; Litorowicz, 2006; Poon et al., 2001; Valenza and Scherer, 2007) or chemical phenomena (ingress of aggressive fluids, gases and ions, eg, sulphate, acid, seawater) occurring within or through the concrete surface (Bai et al., 2003a,b; Brown et al., 2004; Hobbs and Taylor, 2000; Muralidharan et al., 2005). Concrete is porous by nature. Some aspects of concrete durability involve the penetration of certain forms of aggressive agents. Hence the durability of concrete is mostly associated with its permeability or/and diffusion (Verbeck, 1982; Mehta and Monterio, 1993). Fig. 16.1 shows the nature of concrete durability. Inadequate durability can be due to external factors or/and causes within the concrete. The ability to resist the flow of fluids through the porous medium is primarily governed by microclimate parameters (aggressive agents, temperature, and pressure etc.), concrete quality, and transport mechanisms. The intrinsic properties of concrete affecting permeation are the porosity of hardened cement paste (Leemann et al., 2006; Swider et al., 2007), relative size of the pores (Pradhan et al., 2005), and degree of continuity of these pores (Holly et al., 1993). Higher water to binder ratios (w/b) result in an increase in pore volume, hence an increase in permeation (Neville, 2011). Research showed that pozzolanic materials can refine the porosity and pore size distribution of cement paste (Bai et al., 2003a; Khatib et al., 2013a,b; Puentes et al., 2014; Sabir et al., 2001; Yu and Ye, 2013). Fig. 16.2 shows primary and secondary parameters affecting permeation of concrete; these parameters can have an adverse effect on the durability of the concrete. Some excellent references are readily available on transport mechanisms of gases, fluids, and ionics in concrete (Neville, 2011; Nilsson, 1996; Zaharieva et al., 2003). Basheer et al. (2001) gave a comprehensive review on assessment of the durability of concrete

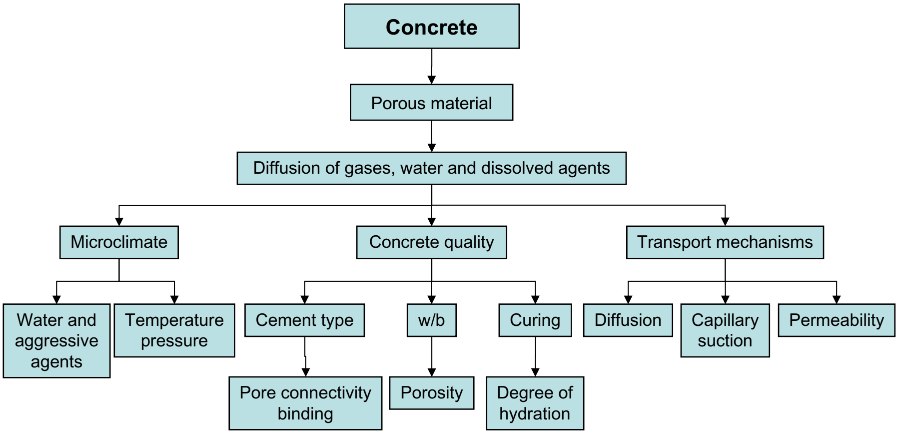

Fig. 16.1 Nature of concrete durability (w/b—water to binder ratio).

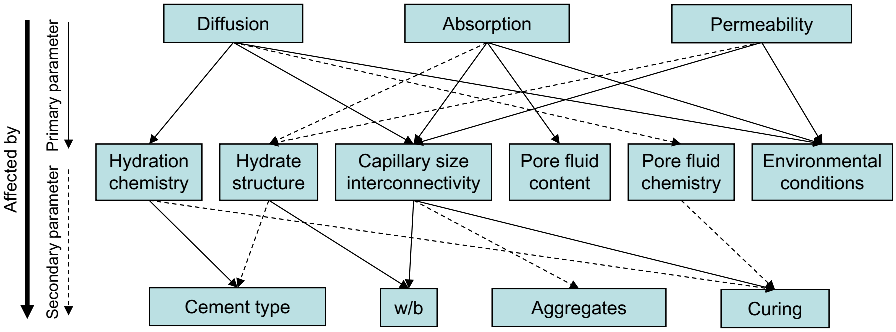

Fig. 16.2 Complexity of concrete durability.

from its permeation properties. Recently, Tang et al. (2015) reviewed major durability problems and especially discussed durability issues in a marine environment.

16.3 Durability of sustainable construction materials

There are various aspects of durability of sustainable construction materials. This section examines the major durability problems in concrete including physical and chemical effects, such as freeze–thaw resistance, cracking, sulphate attack, and chloride-induced degradation.

16.3.1 Freeze–thaw resistance

Various concepts have been proposed to explain the deterioration of concrete materials due to frost action. It is now generally recognised that there are two types of deterioration due to freezing and thawing cycles—internal cracking and surface scaling (Litvan, 1980; Powers, 1945, 1975). Powers (1945) developed a well-known hydraulic pressure theory attributing frost damage in cement paste to stresses caused by hydraulic pressure in the pores. As the water in the capillary pores freezes, there is an increase in volume. If there is no room for the ice to expand into since the pores are full of water, the expansive forces will cause disruption of the pores (Cai and Liu, 1998). If the water within the pores contains dissolved salts in varying concentrations, it means that not all the water will freeze at the same time. These differences in ionic concentrations result in the formation of osmotic pressures (Litvan, 1972; Powers and Helmuth, 1953). They consider that the water adsorbed on the surface or contained in the smaller pores cannot freeze due to the interaction between the surface and the water. Because of the difference in vapour pressure of this unfrozen and much cooled liquid and the bulk ice in the surroundings of the paste system, there will be migration of water to locations where it is able to freeze, such as the larger pores or the outer surface. The process leads to partial desiccation of the paste and accumulation of ice in crevices and cracks. The freezing could also form a semiamorphous solid (noncrystalline ice), resulting in great internal stresses and ultimately breaking up.

Freezing of pore solution in concrete exposed to a freeze–thaw cycle was studied by following the change of concrete electrical conductivity with freezing temperatures.

The way to prevent the internal cracking and disruption that can be due to freezing and thawing cycles is to add air-entraining admixtures to the concrete resulting in the formation of small discrete air bubbles within the capillary network (Du and Folliard, 2005; Gokce et al., 2004; Łaźniewska-Piekarczyk, 2013a). The air voids act as reservoirs into which the ice can expand without causing disruption of the capillaries. This has been confirmed as the most effective way of minimising the damage from frost attack, by laboratory data and field experience (Neville, 2011). Proper air-entrainment, with appropriate volume and spacing factor, will dramatically improve the durability of concrete exposed to moisture during cycles of freezing and thawing (Łaźniewska-Piekarczyk, 2013b,c). Entrained air also improves concrete's resistance to surface scaling caused by chemical deicers (Valenza and Scherer, 2007).

Yüksel et al., (2007) reported that the concrete produced by using granulated blast furnace slag (GBFS) and bottom ash (BA) as partially fine aggregate is more durable to freezing and thawing than reference concrete. About 20–30% replacement ratios can be accepted as optimum replacement ratios. Gokce et al. (2004) reported that freeze–thaw durability of concrete produced from fine materials of waste concrete aggregate was higher than that of concrete produced from normal sand. Hardened concrete incorporating PFA or GGBS has similar or better performance except at early age compared with Portland cement CEM I (BCA, 2002; Dhir et al., 1998; Wang et al., 2013).

Over the past two decades, the use of recovered paper by the UK paper industry has gradually increased to an annual amount of about 5 million tonnes. Wastepaper sludge (which comprises an intimate mixture of finely divided calcium carbonate and kaolin), when heated rapidly (3–5 s) up to a maximum temperature of 1200 °C and then cooled rapidly (3–5 s) to 200 °C, forms a fine ash (WSA), which has cementitious properties (Bai et al., 2003a). Fig. 16.3 shows that PC control mixture failed (over

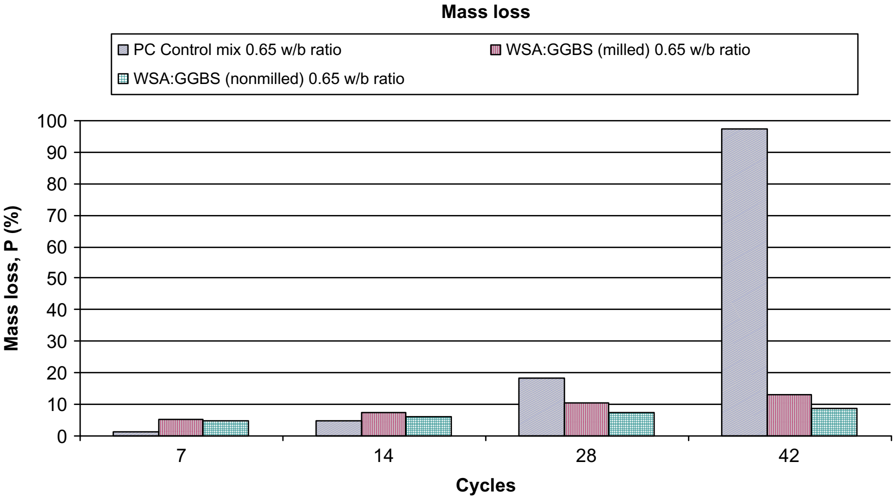

Fig. 16.3 Mass loss of PC control, WSA–GGBS milled and unmilled mixtures under 7, 14, 28, and 42 freeze–thaw cycle attacks (Bai, 2007).

90% loss of mass) under freeze/thaw attack before the 42nd cycle, whereas unmilled and milled wastepaper sludge ash (WSA)–GGBS mixes performed well. Unmilled mix has slightly higher resistance to freeze–thaw than milled mixture.

Edwards (1991) and Flerchinger et al. (2013) reported that frost leads to breakdown of concrete and formation of soil aggregates. Porosity has a major influence on the freeze–thaw durability of stone and aggregates. The total pore volume, the pore size and distribution, and the degree of saturation all contribute to freeze–thaw durability (Litvan, 1980). A rock that has less than 1% water absorption is generally considered to be frost resistant. It is, however, the finer pores, measuring about 1 μm or less, that are most important for the frost resistance of rocks (Lindvall, 2007).

Gokce et al. (2004) reported freeze–thaw durability of concrete produced from waste concrete aggregates with air content to be less durable, but freeze–thaw durability of concrete produced from fine materials of waste concrete aggregates was higher than that of concrete produced from normal sand.

Fibres can provide an adequate internal restraining mechanism and make concrete system perform its crack control function. So randomly distributed fibre in concrete mixture restrains tensile stress due to volume changes of freeze water, and reduces the freeze–thaw damage to concrete.

Exterior masonry walls are exposed to severe weather conditions, such as freeze-thaw action, which affect the durability of masonry units. As the water in the moist unit freezes, it produces pressure in the pores of the unit. If the pressure developed exceeds the tensile strength of the unit, the cavity will dilate and rupture. The cumulative effect of successive freeze–thaw cycles and disruption of the unit can eventually cause expansion and cracking, scaling, and crumbling (Canbaz and Albayrak, 2014). The frost effect on the durability of the units is affected by the moisture content of the units at the start of freezing, the rate of freezing and thawing, and the number of freeze-thaw cycles. It is important that brick possess good durability. One method to improve freeze–thaw characteristics of brick is to increase the compactness of the brick and thus reduce the quantity and diameters of opores (Zhao et al., 2009). Another method is to optimise the formulation and curing regime to obtain hydration products of better freeze–thaw characteristics. Masonry units must comply with the European standard (BSI, 2016) specifying the characteristics and performance requirements for masonry units manufactured from clay for use in masonry construction for building and civil engineering. The appropriate materials subjected to severe exposure conditions should be selected and protected properly, for example, consideration of cappings, copings, and sills in areas where freezing conditions can occur. Earth-retaining walls where saturation with freezing will occur can be considered as being subject to severe exposure.

16.3.2 Abrasion resistance

Abrasion can cause concrete surfaces to wear away, which can be a particular problem in industrial floors. Some hydraulic structures can be made from materials ranging from large rocks and concrete to wooden timbers or tree trunks (Dhir et al., 1991).

The wearing resistance of concrete is the ability of its surface to resist being worn away by rubbing and friction (Horszczaruk, 2005). Wearing of floors and pavements can result from production operations or foot or vehicular traffic. The action of the abrasive materials carried by water on hydraulic structures leads to erosion (ACI 210, 1998). Wearing resistance is closely related to compressive strength at the wearing surface (Dhir et al., 1991). As the paste wears, the fine and coarse aggregates are exposed, and abrasion and impact will cause additional degradation that is related to aggregate-to-paste bond strength and hardness of the aggregate.

Concretes with fine recycled aggregates have greater abrasion resistance than the normal concrete (Evangelista and de Brito, 2007). This may have to do with the fact that abrasion resistance is deeply connected with the bond of the cement paste with the fine aggregates, which is better when recycled aggregates are used. Brito (2005) observed replacing coarse natural aggregates with recycled aggregates of ceramic origin achieved better abrasion resistances as the replacement ratio increased.

Yen et al. (2007) indicated that the concrete mixtures up to 15% cement replacement by fly ash had a similar abrasion–erosion resistance to concrete without fly ash. Beyond 15% cement replacement, the abrasion–erosion resistance decreased.

Concrete with fibres shows higher abrasive resistance (Horszczaruk, 2005). It is 30% higher than the resistance of the nonreinforced concrete. Felekoğlu et al. (2007) reported that steel fibre (156 kg/m^3) addition decreased weight loss due to abrasion by 42% and improved the 28-day flexural strength by 19%.

16.3.3 Cracking

Cracking is a challenge for the concrete industry. Materials selection, environmental conditions, and field practices all have considerable influence on the tendency for early-age cracking to occur (Barluenga and Hernández-Olivares, 2007; Torben and Hansen, 1968). Cracks occur either in plastic concrete or hardened concrete and causes of cracking are many (Fig. 16.4). Plastic shrinkage cracking occurs in fresh concrete

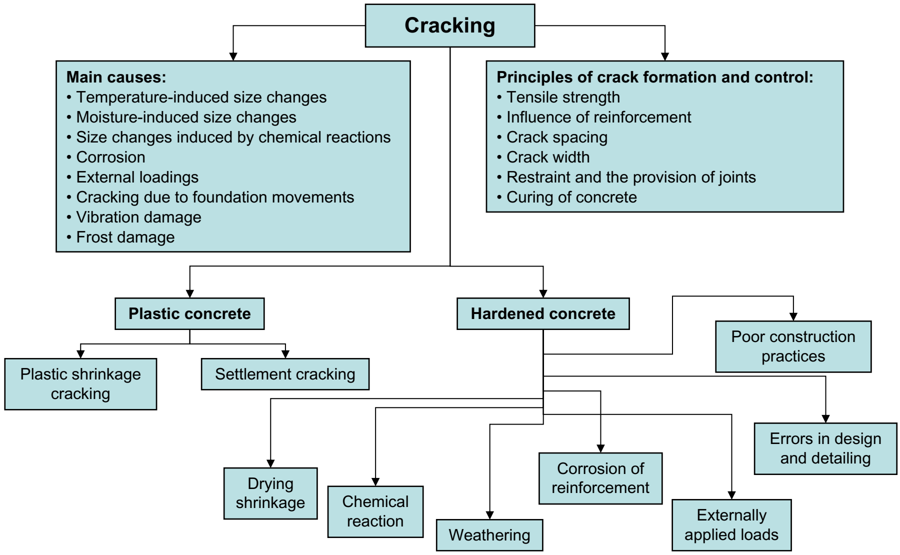

Fig. 16.4 Causes of cracking.

when it is subjected to a very rapid loss of moisture caused by a combination of factors that include air and concrete temperatures, relative humidity, and wind velocity at the surface of the concrete (Cohen et al., 1990). During concrete consolidation period, the plastic concrete may be locally restrained by reinforcing steel or formwork. This local restraint may result in voids and/or cracks (settlement cracking) adjacent to the restraining element (Kronlöf et al., 1995). The weathering processes that can cause cracking include freezing and thawing, wetting and drying, and heating and cooling. Size changes induced by chemical reactions could be very disruptive. Cracking initiated by external loading and reinforcement corrosion could lead to catastrophic failures of concrete structures (Fig. 16.5).

To prevent and minimise cracks in concrete, it is important to identify the causes of cracks (Abrishambaf et al., 2015; Šavija et al., 2013). The major factors controlling ultimate drying shrinkage of concrete, for example, include relative humidity, aggregate type and content (or paste content), water content, and water to cement ratio. The rate of moisture loss and shrinkage of a given concrete is influenced by the size of concrete member, the relative humidity, distance from the exposed surface, and drying time. The disruptive character of cracking from some kinds of chemically induced size changes potentially impairs the life and the structural serviceability of affected construction. These likely include corrosion of metals, sulphate attack, hydration, and carbonation. Designers, specifiers, and surveyors should therefore be particularly vigilant in identifying the risks associated with the use of recycled materials. In general, the design strategy would be to avoid the use of susceptible materials or of susceptible combinations of materials, to ensure that service conditions are not favourable

Fig. 16.5 Cracking due to loading and reinforcement corrosion.

to reactions and to protect susceptible materials, where appropriate, from conditions favouring reaction. One possible solution to reduce the impact of early age shrinkage on concrete durability is to include low volumetric fractions of short fibres in order to control crack growth.

The need for repair and strengthening of deteriorated and damaged infrastructures due to cracking has become an important challenge worldwide. To meet the growing need for concrete repair and rehabilitation, fibre-reinforced polymer (FRP) systems (Bai, 2013) for strengthening concrete structures are unique to traditional strengthening techniques, such as steel plate bonding, section enlargement, and external post-tensioning. FRP systems offer advantages over traditional strengthening techniques in that, they are lightweight, relatively easy to install, and noncorrosive.

16.3.4 Alkali-silica reaction

The alkali-silica reaction has received greatest attention and was the first to be recognised. It involves a reaction between the OH^- ion associated with the alkalies (Na_2O and K_2O) from the cement and other sources, with certain siliceous constituents that can be present in the aggregate. This phenomenon was referred to as alkali-aggregate reaction but is more properly designated as alkali-silica reaction. Alkali-silica reaction adversely affects the appearance and serviceability of a structure. The reaction is expansive leading to eventual spalling of the concrete cover and internal cracking (Vidal et al., 2007).

Gress et al. (2000) carried out extensive research on techniques and procedures to assess the alkali silica reactivity (ASR) expansion potential for concrete made from recycled concrete aggregate (RCA). Expansion of concrete cubes was found to be significantly accelerated as compared to that of a standard prism. Sealing prisms in evacuated plastic bags with water was also found to accelerate ASR expansion effectively.

Glass is unstable in the alkaline environment of concrete and could cause deleterious alkali-silica reaction problems. Taha and Nounu (2008) reported that glass powder as cement replacement used in concrete as a pozzolanic amorphous material can contribute to the formation of hydration products and C–S–H gel. Moreover, the results of this study showed that when pozzolanic glass powder was used in concrete as cement replacement, the ASR expansion was significantly reduced even with high alkali content. The presence of pozzolanic glass powder in concrete as cement replacement leads to changes in the concentration of hydroxide ion OH^- in the pore solution, which is considered as a direct reason in reducing the risk of ASR expansion.

Not all aggregates are susceptible to deleterious ASR. However, selection must avoid aggregates containing high levels of reactive minerals or rocks. Use of pozzolanic materials is an effective option to minimise the potential for the deleterious effect of ASR as this can reduce the pH values of the concrete pore solution.

16.3.5 Sulphate attack

Sulphate attack on concrete is the most common form of chemical attack. Sulphates are commonly found in soil, aggregates, seawater, and cements. The chemical consequences of sulphate attack on concrete components are (Hewlett, 2003; Hobbs and Taylor, 2000; Tang et al., 2015):

- the formation of ettringite (calcium aluminate trisulphate), resulting in an increase in solid volume, leading to expansion and cracking

$$2\left(3CaO \cdot Al_2O_3 \cdot 12H_2O\right) + 3\left(Na_2SO_4 \cdot 10H_2O\right)$$
$$\Rightarrow 3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 31H_2O + 2Al\left(OH\right)_3 + 6NaOH + 17H_2O$$

- the formation of gypsum (calcium sulphate dihydrate), leading to softening and loss of concrete strength

$$Ca\left(OH\right)_2 + Na_2SO_4 \cdot 10H_2O \Rightarrow CaSO_4 \cdot 2H_2O + 2NaOH + 8H_2O$$

Protection against sulphate attack can be achieved by using concrete with low permeability, using cements with a low C_3A content and blends of PC with pozzolans (Khatib et al., 2008; Mangat and El-Khatib, 1992; Mangat and Khatib, 1995) (thus reducing the available $Ca[OH]_2$). Proper placement, compaction, finishing, and curing of concrete are essential to minimise the ingress and movement of water, which is the carrier of the aggressive salts. Krammart and Tangtermsirikul (2004) indicated that the expansions of municipal solid waste ashes and calcium carbide waste cements in sodium sulphate solution were lower than that of the control cement when exposed to sulphate solution.

There are many factors affecting sulphate attack and steps to mitigate it. In general, three approaches are adopted: (a) preventing sulphates from penetrating into concrete; (b) consuming $Ca(OH)_2$ as much as possible in hydrated cement matrix through the use of pozzolana such as PFA and GGBS; and (c) using cement with low C_3A.

The first approach is very important and can be achieved by producing impermeable concrete so as to stop or slow the penetration of sulphates into concrete, thus to extend the concrete service life. For concrete structures which are in contact with sulphate-bearing soils, protective linings such as the various proprietary self-adhesive membranes, or protective coatings, such as bitumens, tars and epoxy resins, may also be applied on exterior surfaces.

Pozzolans such as PFA (Chindaprasirt et al., 2004; Zuquan et al., 2007), silica fume (Ganjian and Pouya, 2005; Roy et al., 2001), MK (Hossack and Thomas, 2015; Khatib and Hibbert, 2005; Sabir et al., 2001) and GGBS (Gollop and Taylor, 1996; Higgins, 2003) can be employed to effect improvement in the resistance of concrete to sulphate attack. The effect of pozzolana is that the pozzolanic reactions consume $Ca(OH)_2$, which is needed for reaction with sulphates. In the meantime, blended cement concrete has less $Ca(OH)_2$ due to the replacement of cement with pozzolana.

Finally, the formation of ettringite can be minimised by using sulphate-resisting cement, which has lower C_3A content. This in turn will reduce the expansive ettringite formation.

16.3.6 Chloride-induced degradation and steel corrosion

It is generally established that hydrated PC in mortar and concrete is susceptible to attack by seawater, which contains both chloride and sulphate. The principal outcomes are a significant loss of strength, due primarily to decalcification of the C–S–H gel, and diffusion of chloride ions into the concrete, which can initiate corrosion of steel reinforcement. Partial replacement of PC by pozzolans such as PFA, GGBS, and MK can retard these processes (Al-Otaibi, 2008; Bai et al., 2003b).

Chlorides can come from a number of different sources, such as deicing salts, unwashed marine aggregates, and seawater. Chloride ions cause localised breakdown of the passive film that initially forms on steel as a result of the alkaline nature of the pore solution in concrete. After initiation of the corrosion process, the accumulation of corrosion products (iron oxides and hydroxides) occupying a volume several times larger than that of the original iron (Pradhan, 2014; Vidal et al., 2007), leads to internal stresses that result in cracking and spalling of the concrete cover.

Resistance to chloride ingress is dependent mainly upon the cement or blended cement with pozzolans and the w/c ratio, with aggregate quality being a secondary factor (Lindvall, 2007). Compressive strength is included as an indirect control on these parameters (BSI, 2013). Through careful design and good construction practices, the protection provided by PC concrete to embedded reinforcing steel can be optimised. Bai et al. (2003b) reported that significant reduction in chloride penetration depths occurs when PC (cement) is partially replaced with PFA and MK. These reductions increase with both increasing total replacement level and increasing exposure time. This is attributed to the relative changes in intrinsic diffusivity and chloride binding capacity with age exhibited by the different binder compositions. The quality and depth of concrete in the cover zone are all important in minimising the risk of corrosion. Khatib and Mangat (2002) investigated the influence of high-temperature and low-humidity curing on chloride penetration in concrete containing cement replacement materials. Higher chloride penetration resistance was observed when cement is partially replaced with either fly ash or silica fume.

Ann et al. (2008) reported that PFA and GGBS were used to compensate for the loss of strength and durability of concrete containing recycled aggregate. As a result, 30% PFA and 65% GGBS concretes increased the compressive strength to the level of control specimens cast with natural granite gravel. Replacement with PFA and GGBS was effective in raising the resistance to chloride ion penetration into the concrete body, measured by a rapid chloride ion penetration test based on ASTM C 1202-91. It was found that the corrosion rate of 30% PFA and 65% GGBS concretes was kept at a lower level after corrosion initiation, compared to the control specimens.

For plain concrete, the presence of chlorides is not generally a cause for concern in the UK. However, for concrete-containing reinforcement, the presence of chlorides is potentially very serious and has been responsible for the damage of many structures.

In most cases, concrete provides very good protection to embedded steel owing to a number of physical and chemical phenomena. It is only under specific conditions that this protection will break down and corrosion occur. Concrete cover in combination with chemical resistance are the main factors influencing the effectiveness of concrete against corrosion initiation. High alkalinity of the concrete pore fluids ($pH > 12.5$), owing to calcium hydroxide (CH) produced from cement hydration, provides conditions that leave embedded steel in a thermodynamically stable or passive and noncorroding condition.

Destruction of the passive conditions involves two processes mainly responsible for occurrence of corrosion: the ingresses of carbon dioxide (carbonation) and chloride ions coupled with the presence of oxygen and water. Under this condition, the ferrous ions (Fe^{2+}) released from the anode combine with the hydroxyl ions (OH^-) from the cathode, in the presence of water and oxygen to produce rust (ferric hydroxide, $Fe[OH]_2$). This is an expansive reaction leading to eventual spalling of the concrete cover and reduction in the area of the steel section at the anodic site. The depth of concrete cover is important in minimising the risk of steel corrosion.

16.3.7 Efflorescence

Efflorescence is a deposit of salts, usually white, formed on a surface. The substance emerged in solution from within either concrete or masonry and subsequently was precipitated by evaporation. It occurs most readily in porous concrete near the surface. Efflorescence is not normally damaging, but it is aesthetically undesirable. Dow and Glasser (2003) developed a physicochemical model to explain the origin of efflorescence and quantify the key features of its formation. Brocken and Nijland (2004) investigated the efflorescence process and its relevant parameters and noted that efflorescence on masonry is generally formed by (hydrated) Na^-, K^-, Ca-sulphates, or carbonates. Single and double salts commonly encountered are thenardite, Na_2SO_4, glaserite, $K_3Na(SO_4)_2$, and syngenite, $K_2Ca(SO_4)_2{\cdot}H_2O$. In mortar, sulphate generally originates from the calcium-sulphate (gypsum, anhydrite, hemihydrate) added to control setting. Efflorescence of calcium carbonates, notably calcite, $CaCO_3$ (formed by the reaction of $Ca[OH]_2$ with CO_2), often occurs on surfaces of masonry or concrete elements. Efflorescence of gypsum occurs as a white, foggy deposit on the surface of clay bricks. Efflorescence and washout of lime typically occurs on masonry units. During wetting, rain runs off from the masonry surfaces and causes excessive wetting of the mortar joints. This facilitates dissolution of lime in the pore water of the mortar joints and prevents carbonation of this lime.

Efflorescence can be limited or prevented through proper drainage, and concrete sealers can prevent efflorescence from reoccurring. Use of lime and GGBS in mortar mixtures would effectively eliminate efflorescence in masonry structures.

16.4 Future trend

Though significant research on the durability of concrete has been carried out over the last a few decades, concrete durability is still a very hot topic due to the advances in incorporating sustainable SCMs in concrete. This chapter has reviewed and examined current research achievement on various aspects of the durability of concrete. This section presents future trends in research development of concrete technology.

It is now accepted that sustainable SCMs can be used to enhance concrete performance and extend the life cycle of concrete structures (Bai and Wild, 2002; Bai et al., 2003b; Hunag et al., 2015; Khatib and Hibbert, 2005; Mangat et al., 2006; Merida and Kharchi, 2015; Sabir et al., 2001). Understanding of the role of SCMs in concrete needs to be further advanced. To some extent, the type and amount of pozzolanic materials in concrete require to be optimised for cost efficency and improvement of the durability performance.

The planet faces a range of vulnerability scenarios due to greenhouse gas emissions. The UK government is committed to reducing CO_2 emissions by 80% compared with the 1990 baseline level by 2050 (UK Parliament, 2008). Almost half of these emissions were coming from the building stock (BRE, 2007). Recycling of waste reduces the demand for virgin materials (Cimpan et al., 2015a), the amount of waste to be incinerated, and/or landfilled (Cimpan et al., 2015b), and also reduces emissions from these sources (Singer et al., 2014). It is equally important to produce thermally and energy efficient concrete with the recycled waste. Therefore concrete mixture optimisation method is desirable for sustainable construction with purposed materials.

Finally, the consideration of durability in concrete structure design needs to be integrated with the selection of suitable sustainable concrete materials. The purposed materials are selected with consideration of the effect of mechanical actions/loads and environmental factors and service life prediction based on durability.

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Low clinker cement as a sustainable construction material

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Global clinker production in 2014 stood at 3.57 Bt (US Geological Survey, 2015a). With the production of 1 tonne of Portland cement clinker resulting in carbon dioxide emissions of about 900 kg, this level of production means that the global cement industry is responsible for 6–7% of anthropogenic carbon dioxide emissions. Consequently, the industry is looking for ways to reduce its carbon footprint and for reducing the clinker content within cement; for example, the clinker factor is an effective means for doing so.

While CEM I cement may, according to EN197-1, contain up to 5% of minor additional constituents, it is the addition of supplementary cementitious materials (SCMs) which reduces the clinker factor considerably. EN197-1 details five classes of SCMs which may be added to Portland cement clinker to make what may be called composite cements. These five materials are ground granulated blast-furnace slag (GGBFS), natural pozzolans, fly ashes, burnt shale and silica fume.[1] In all cases, the materials are slightly lower in calcium than Portland cement is, as shown in Fig. 17.1.

Each of these materials is latently hydraulic or pozzolanic, that is, inert in itself but reactive in the presence of alkalis. As such, with the addition of water, these materials all react to produce hydrated phases which help to improve the technical performance of the composite cement. Furthermore, being industrial by-products or less energy-intensive materials, these materials all help to reduce the environmental footprint of cement. This chapter will consider each of these classes of SCMs in turn, plus selected others, looking first at their origins, composition and physical properties. It will then look at how each SCM will affect cement hydration, the impact on durability and the environmental benefits of SCMs. Finally, the chapter will look at how changes in production techniques and the supply of materials may affect future availability and performance of SCMs in general.

17.1 Ground granulated blast-furnace slag

17.1.1 Origin of GGBFS, composition and physical properties

In a blast furnace, iron ore, fuel (often coke) and flux (often limestone) are continuously supplied from the top of the furnace, while blasts of hot air are blown into the lower part of the furnace. At temperatures of approximately 1500°C, silicate and aluminate impurities from the ore and fuel combine with the flux to form a molten slag. This slag, being less dense than the molten iron, floats and can be tapped off.

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[1] Note that the standard also lists limestone as a permissible component, but it is not considered here.

Sustainability of Construction Materials. http://dx.doi.org/10.1016/B978-0-08-100370-1.00017-2

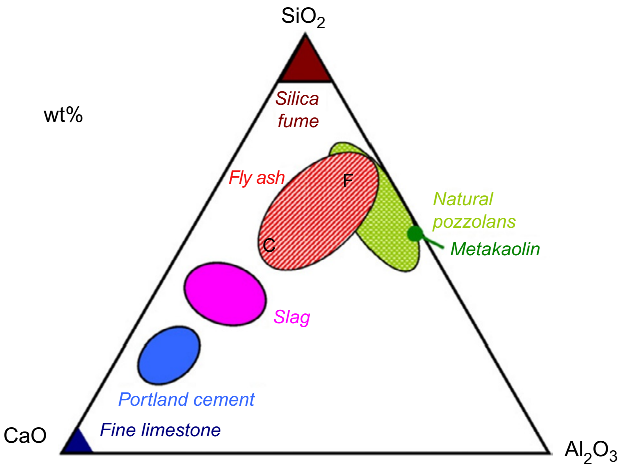

Fig. 17.1 CaO–Al_2O_3–SiO_2 ternary diagram of cementitious materials (Lothenbach et al., 2011).

If the slag is allowed to cool slowly, then a stable crystalline material is obtained, which can be used as aggregate in concrete or road construction but is not a suitable addition to cement. However, rapid cooling to below 800°C, either by spraying with water or via a semidry granulation process, avoids crystallisation and leads to the formation of a glassy material. The glassy, granular material is then ground to a similar fineness as cement clinker, yielding a light grey powder commonly known as GGBFS. The slag has a relative density of 2.85–2.95, with a bulk density of 1050–1375 kg/m^3. Precise figures for global production of ground granulated blast-furnace slag are difficult to find, but 2014 production was estimated to be 310–370 million tonnes (US Geological Survey, 2015b), that is, about 10% of global cement production.

GGBFS is typically ground to 400–600 Blaine, and a typical particle size distribution is shown in Fig. 17.2. Particle sizes range from about 0.5 μm to almost 100 μm, with finer particles reacting more quickly such that those particles with a diameter less than 10 μm contribute to early age strength, that is, up to 28 days. Particles 10–45 μm in diameter contribute more to later-age strength, whereas particles larger than 45 μm are considered to be unreactive. The British Standard for GGBFS (BS EN 15167-1, 2006) stipulates that the glassy content of GGBFS must be at least two-thirds of the weight. However, glass contents in excess of 90% are typical, with small quantities of gehlenite (C_2AS), akermanite (C_2MS_2) and merwinite (C_3MS_2) being the most common crystalline phases present. Fig. 17.3 shows typical X-ray diffraction (XRD) patterns for GGBFS, with a broad hump centred at just above 30 degree 2θ (CuKα source), together with reflections for gehlenite and calcite.

Table 17.1 shows the elemental composition of a range of GGBFS samples. As can clearly be seen, the composition of GGBFS is not fixed, but rather varies depending on the composition of the iron ore, fuel and limestone flux. However, slags are characterised by comprising primarily lime, silica, alumina and magnesia. Compared to Portland cement clinker, slags are typically low in lime and enriched in silica, alumina and magnesia. BS EN15167-1 stipulates that GGBFS must comprise, by mass, more

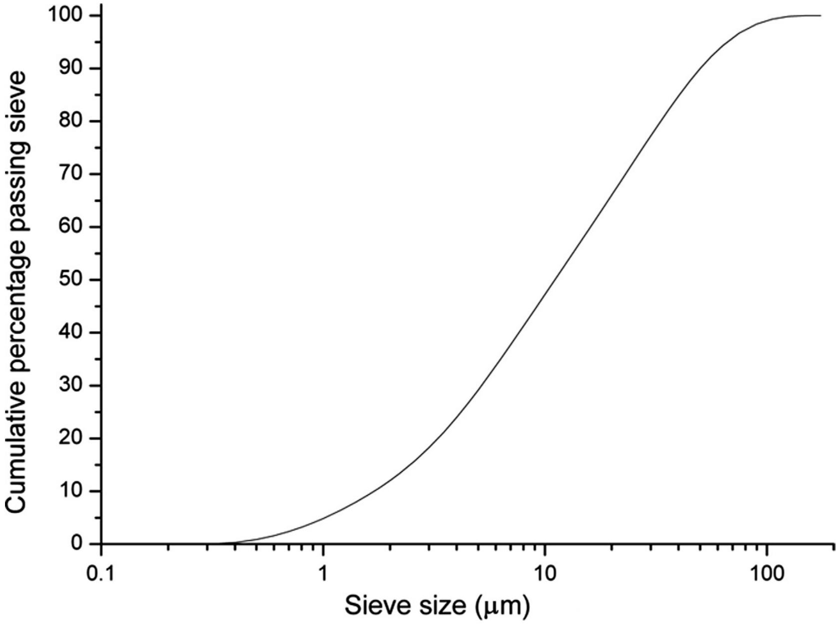

Fig. 17.2 Typical particle size distribution of GGBFS.

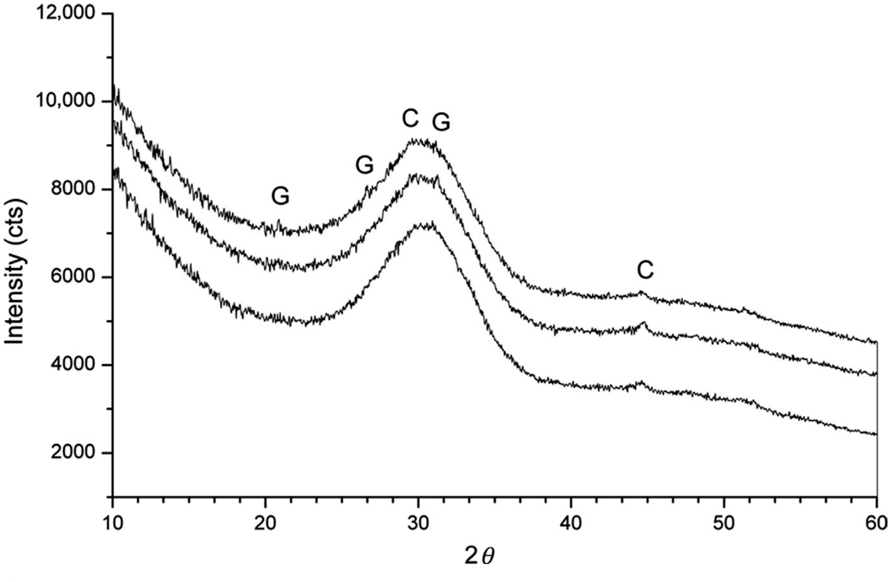

Fig.17.3 XRD pattern for a typical GGBFS sample (G = Gehlenite; C = Calcite).

Table 17.1 Chemical compositions and basicity indices for a range of typical GGBFS samples

	Canada (Ramezanianpour and Hooton, 2012)	**UK**	**Japan (Higashiyama et al., 2014)**	**China (Yi et al., 2015)**
Element				
CaO	39.95	39.76	43.1	34.00
Al_2O_3	7.18	10.81	15.0	17.90
SiO_2	38.14	37.15	33.8	34.30
MgO	10.57	7.86	5.63	6.02
Fe_2O_3	0.74	0.41	0.27	1.02
SO_3	2.71	1.66	1.93	1.64
Basicity index				
P_1	1.05	1.07	1.28	0.99
P_2	1.32	1.28	1.44	1.17
P_3	1.11	0.99	1.00	0.77
P_4	1.54	1.53	1.76	1.53
P_5	1.51	1.57	1.89	1.69

than two-thirds calcium oxide (CaO), magnesium oxide (MgO) and silicon dioxide (SiO_2), whereas the ratio by mass $(CaO+MgO)/(SiO_2)$ must be greater than 1.0.

17.1.2 Hydraulic properties of slag cements

GGBFS is a latently hydraulic material such that water will induce a hydraulic reaction, but the reaction is very slow. The reaction may be accelerated by the addition of alkali, for example, portlandite formed when Portland cement is hydrated.

A number of factors influence the reactivity of GGBFS, including fineness, crystallinity, chemical composition, the replacement level and the nature of the activator. Finer materials, with a larger specific surface area, react more quickly. Similarly, the greater the glass content of the slags, the greater the reactivity.

Numerous attempts have been made to relate slag composition to reactivity. Various basicity indices have been proposed (Table 17.2), each of which consider the relative proportions of basic (Ca, Mg) and acidic (Al, Si) elements. A number of these indices are shown in Table 17.2 for a range of slags of different compositions. While it is generally accepted that basic slags (ie, where Ca/Si > 1.0) are more reactive, the relationship between composition and reactivity is not clear. Despite this, such indices do find their way into standards, with the British standard stipulating Ca/Si > 1.0. Then, as with measures of pozzolanicity, an activity index can be determined by comparing the compressive strength of a 50:50 GGBFS–cement mortar to that of a reference cement mortar. BS EN 15167-1 stipulates that ‘the activity index at 7 days and at 28 days shall be not less than 45% and 70%, respectively’.

When blended with cement, slag dissolution is induced by the high pH of the solution, caused by the calcium hydroxide and alkali ions in the pore solution. Both the clinker and the slag react alongside one another producing calcium silicate hydrate

Table 17.2 Basicity ratios proposed to define GGBFS reactivity

Basicity ratio	Requirement for good performance
$P_1 = \frac{CaO}{SiO_2}$	>1.0
$P_2 = \frac{CaO + MgO}{SiO_2}$	>1.0
$P_3 = \frac{CaO + MgO}{SiO_2 + Al_2O_3}$	1.0–1.3
$P_4 = \frac{CaO + 0.56Al_2O_3 + 1.4MgO}{SiO_2}$	>1.65
$P_5 = \frac{CaO + MgO + Al_2O_3}{SiO_2}$	>1.0

After Smolczyk, H.G., 1980. Slag structure and identification of slags. In: Proceedings of the 7th International Congress on the Chemistry of Cement, Paris, France. vol. I, pp. III-1/3–1/17.

(C-S-H), thus leading to strength development. Hydration of GGBFS is typically more gradual than it is with Portland cement. Additionally, during the very early stages of hydration, the presence of slag may be considered, in part, as an inert filler. This filler effect promotes the nucleation and growth of C-S-H by increasing the effective water/cement ratio and providing nucleation sites.

Fig. 17.4 shows the degree of slag hydration in a selection of cement–slag blends. The figure shows how both slag composition and replacement level can affect the degree of hydration. Increasing slag basicity leads to greater slag hydration at a given time, whereas greater replacement levels lead to reduced slag hydration, due in part to the reduced levels of portlandite from the lower cement content. For comparison, over 80% of the cement hydrated in each system after 7 days (Whittaker et al., 2014). Unlike alkali-activated systems, the use of different cements has little impact on the hydration of the slag.

For the most part, hydration of slag cement leads to the formation of the same phases as for a CEM I pure Portland cement, that is, C-S-H, portlandite, ettringite and AFm phases. In addition, a hydrotalcite phase, and occasionally strätlingite, C_2ASH_8, can form. The dilution of cement leads to reduced levels of portlandite, AFm and ettringite. Furthermore, slag hydration has been shown to consume portlandite, particularly at later ages. Meanwhile, aluminium from the slag can be released upon hydration, leading to slightly elevated AFm levels, although the dilution effect and reduced rate of slag hydration mean that the increase in AFm content is very slight.

Slag addition also has a slight yet significant impact on the C-S-H formed. Increasing replacement levels lead to a C-S-H phase increasingly depleted in calcium,

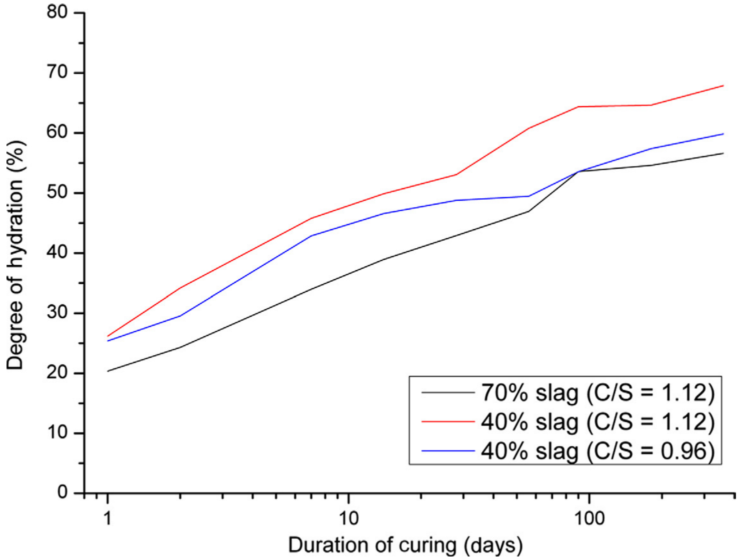

Fig. 17.4 Degree of hydration of GGBFS as a function of time.
After Whittaker, M., Zajac, M., Ben Haha, M., Bullerjahn, F., Black, L., 2014. The role of the alumina content of slag, plus the presence of additional sulfate on the hydration and microstructure of Portland cement-slag blends. Cem. Concr. Res. 66, 91–101.

with the incorporation of small quantities of aluminium. Furthermore, there is a change in C-S-H morphology with increasing slag addition, from fibrillar to foil-like (Richardson, 2008). This contributes to the decreased permeability of slag cements.

17.1.3 Effect of slag on concrete performance

The smooth surfaces of GGBFS particles mean that replacement of cement clinker with slag can improve the workability of concrete, although replacement appears to have no significant impact on bleeding and segregation. Setting times, meanwhile, may be affected by slag replacement. The aforementioned lower reactivity of slags compared to cement clinker means that final setting times are always delayed, particularly at lower temperatures and higher replacement levels. Initial setting times meanwhile may be delayed, remain unchanged, or even shorten depending upon replacement levels. At low replacement levels (up to c.25%), the filler effect promotes clinker hydration, which compensates for the lower rate of slag hydration such that the initial setting time remains unchanged or is shortened. At higher replacement levels, final setting times are delayed, particularly at lower temperatures.

Strength development is more gradual in slag cements than in CEM I systems. This is due to the lower reactivity of the slag than of the clinker. However, typically by 28 days, slag cements have developed comparable strengths to equivalent CEM I systems, whereas for longer periods, they may exhibit higher strengths. As with clinker, the fineness of the slag impacts compressive strength development. Finer slags, having higher specific surface areas, react more quickly, thus contributing greatly to early age strength. However, it is still the clinker which makes a greater contribution to early age strength, whereas the slag contributes more to later-age strength.

As with CEM I systems, strength development may be affected by changes in temperature and relative humidity. The lower activation energy of slag means that its rate of hydration is greatly accelerated at elevated temperatures. Fig. 17.5 shows the strength development of a 70:30 cement–GGBFS blend at both 20°C and 38°C (Ogirigbo and Black, 2015). Higher temperature curing leads to elevated early age strength, but lower later-age strength.

As with many SCMs, the slower hydration of slag may necessitate longer curing times for concretes containing high replacement levels. In many standards, minimum curing times are defined by the ratio of early age to late-age strength, with a basic principle being that curing must be at least long enough for the component to reach 50% of its characteristic strength f_{ck}. For example, DN1045-3 uses the ratio of the average 2- to 28-day compressive strengths at 20°C, which leads to classifications in the ranges of rapid, average, slow and very slow strength development. Minimum curing durations are then further defined depending upon the surface temperature, as shown in Table 17.3.

Such an approach requires compliance testing of concrete samples of comparable composition, and does not explicitly recognise the effects of replacement type, level or composition. Guidelines from the Japan Society of Civil Engineers (JSCE) (Numata and Tsuji, 1996) also recommend moist curing durations sufficient to achieve 50% of the 28-day compressive strength (for 50% replacement and w/b of approximately 0.5). However, the recommendations go one step further and define minimum curing times based on temperature, replacement level and slag fineness, but not slag composition.

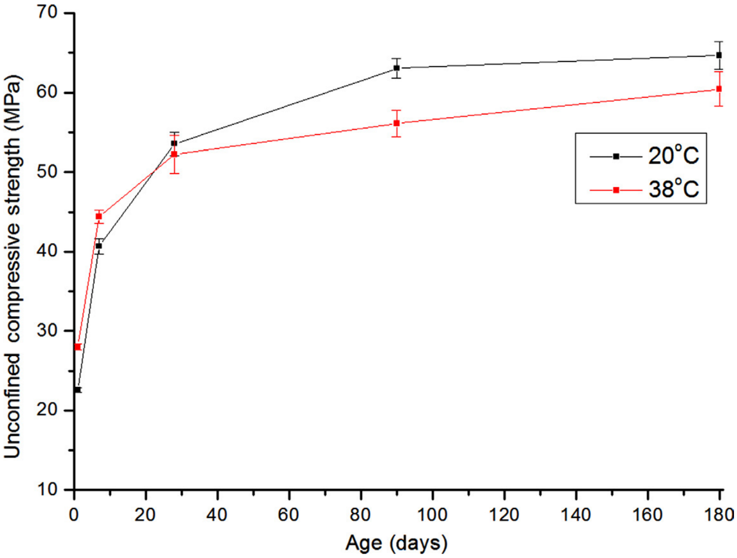

Fig. 17.5 Strength development of 70:30 cement–GGBFS mortars at 20°C and 38°C (Ogirigbo and Black, 2015).

Table 17.3 Minimum curing duration for concrete complying with DIN 1045-3 for all exposure classes with the exception of X0 and X1

Strength development of the concrete	Rapid	Average	Slow	Very slow
$r = f_{cm2}/f_{cm28}$	$r \geq 0.50$	$r \geq 0.30$	$r \geq 0.15$	$r \leq 0.15$
Surface temperature (θ) in °C	Minimum curing time in days			
$\theta \geq 25$	1	2	2	3
$25 \geq \theta \geq 15$	1	2	4	5
$15 \geq \theta \geq 10$	2	4	7	10
$10 \geq \theta \geq 5$	3	6	10	15

After DIN 1045-3, 2012. Tragwerke aus Beton, Stahlbeton und Spannbeton—Teil 3: Bauausführung- Anwendungsregeln zu DIN EN 13670. Concrete, reinforced and prestressed concrete structures—part 3: execution of structures—application rules for DIN EN 13670.

As Table 17.4 shows, the guidelines require longer curing periods with higher replacement levels, lower temperatures and coarser slag.

17.1.4 Durability of slag cements

Providing that there has been adequate curing, the hydration of slag within cement blends leads to a modification of the microstructure of the hardened cement paste. The shift from a fibrillar to a foil-like C-S-H morphology (Richardson, 2008) together with a reduction in capillary porosity will reduce the permeability. This will help to prevent the ingress of aggressive species, thus potentially improving durability.

Table 17.4 **Standard minimum period of moist curing of slag cements according to JSCA (Numata and Tsuji, 1996)**

Replacement ratio (%)	30–40	50			55–70
Specific surface area (cm^2/g)	3000–5000	3000–5000	5000–7000	7000–9000	3000–5000
Daily mean temperature (°C)	Duration of moist curing (days)				
17	6	7	7	6	8
10	9	10	9	8	11
5	12	13	12	10	14

17.1.4.1 Resistance to carbonation

The resistance of concrete to an attack by atmospheric carbon dioxide is an important consideration for reinforced concrete. The consumption of portlandite during slag hydration appears to dominate over the reduction in permeability such that under laboratory conditions, the resistance of many slag cement systems to carbonation has been found inferior to equivalent cement systems. Furthermore, carbonation of slag cements may lead to slight coarsening of the pore structure, further undermining carbonation resistance. Ngala and Page (1997) investigated the change in capillary and total porosities upon carbonation of Portland cement pastes, plus 65% GGBFS and 30% PFA blends. All systems showed a decrease in total porosity upon carbonation. However, while the pure cement system showed no change in capillary porosity, this increased upon carbonation of the composite cements (Fig. 17.6).

However, many laboratory carbonation studies (including those performed by Ngala and Page) are conducted under conditions which maximise carbonation. In particular, relative humidities are often maintained at about 65%. Under field conditions, where relative humidities may fluctuate, slag cements may often perform better than pure cement

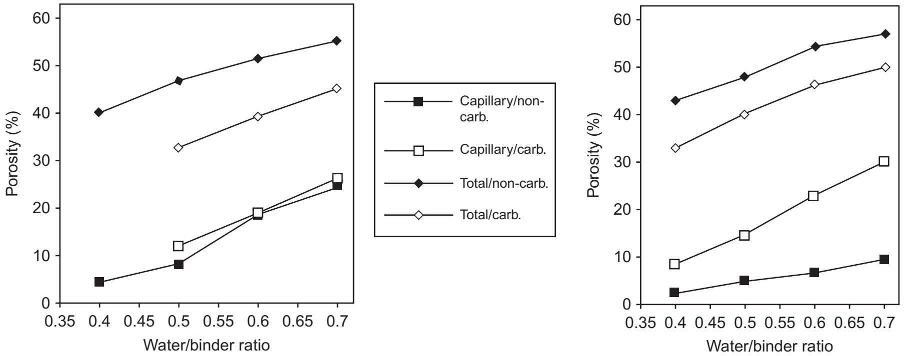

Fig. 17.6 Total and capillary porosities of OPC (*left*) and 65% GGBFS (*right*) blends pre- and post-carbonation (Ngala and Page, 1997).

systems (Kurdowski, 2014). This may be ascribed to their finer pore structures. According to the Kelvin equation, smaller pores become saturated at lower relative humidities such that the finer pore structure within slag composite cements is more often saturated, thus greatly reducing available porosity and markedly reducing transport of carbon dioxide.

17.1.4.2 Resistance to chloride ingress

Penetration of chlorides into reinforced concrete, leading to corrosion of the reinforcement, is a major cause for concern. The finer pore structure of slag cement pastes, leading to reduced permeability, is beneficial in resisting chloride ion ingress. However, slag cements have been shown to improve resistance even when improperly cured. This is due to the chemical composition of the GGBFS, with the elevated aluminate content of the slag binding chlorides as Friedel's salt within the hardened cement matrix. Fig. 17.7 compares the chloride ingress at 20°C into a 30:70 GGBFS–Portland cement composite mortars against that of an equivalent Portland cement mortar. The two slag samples were of slightly different composition, with slag 1 being slightly more basic with a higher aluminium content. Although there was a slight difference between the two composite samples, with the more basic slag performing slightly better, both slag blends performed significantly better than the equivalent CEM I blend. This was attributed to the refined pore structure of a slag blend reducing porosity, whereas the increased aluminium content enabled chloride immobilisation through the formation of Friedel's salt.

17.1.4.3 Resistance to sulphate attack

As with chloride attack, replacement of clinker with GGBFS improves resistance to sulphate attack, so much so that slag cement composites have now pretty much replaced

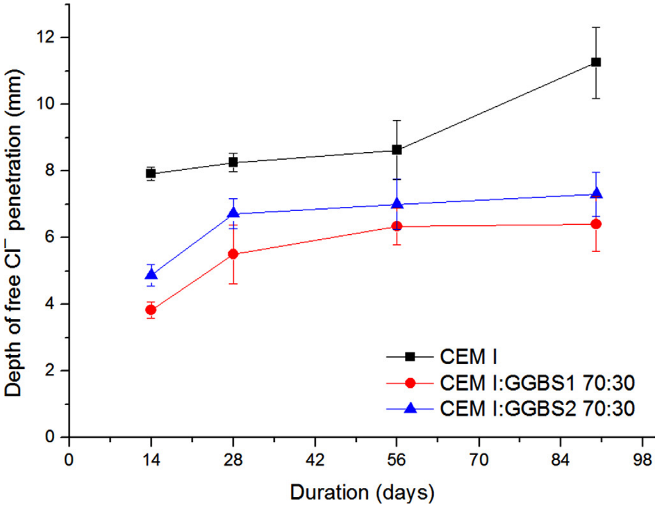

Fig. 17.7 Chloride penetration into slag cements.
After Ogirigbo, O.R., Black, L., 2015. Combined influence of slag composition and temperature on the performance of slag blends. In: 4th International Conference on Concrete Repair, Rehabilitation and Retrofitting (ICCRRR), 5–7 October 2015, Leipzig, Germany.

sulphate-resisting Portland cement for use in sulphate-rich environments. Three reasons have been given for the improved sulphate resistance (Al-Amoudi, 2002):

1. Clinker dilution. Lower C_3A and C_3S contents result in fewer aluminate hydrates and portlandite to react with sulphates.
2. Portlandite consumption during clinker hydration. As such, less portlandite is available to form gypsum or to provide calcium to form ettringite.
3. Lower permeability. Blended systems usually exhibit a finer pore structure, reducing permeability and thereby improving resistance.

The behaviour of cement–GGBFS composites depend on the level of replacement, the composition of the slag and the permeability of the concrete. High replacement levels (c.70%) invariably improve sulphate resistance, whereas the behaviour of systems with lower levels of replacement are variable (Mangat and Khatib, 1992; Mangat and Khatib, 1995). Resistance appears to depend on the alumina content of the slag, with lower alumina levels performing best (Whittaker, 2014).

The use of high levels of slags to improve resistance may appear to be counterintuitive with their higher aluminium content. Aluminium released during hydration is distributed between C-S-H, aluminate hydrates (AFt and AFm) and hydrotalcite, plus any remaining in unreacted slag. Only aluminium bound by the AFm is readily available to react with sulphates. This helps to reduce the effects of sulphate attack. Furthermore, during sulphate attack, alumina within residual anhydrous slag reacts with sulphates to form secondary monosulphate prior to forming secondary ettringite (Fernandez-Altable, 2009). This explains the improved resistance of slag cement composites.

Fig. 17.8 shows linear expansion data for a range of mortar samples immersed in 3 g. L^{-1} sodium sulphate solution for up to 2 years. All of the composite systems performed significantly better than the Portland cement mortar, with the 70% replacement system performing best of all. Of the two 40% replacement systems, there was little difference between a more basic slag (C/S = 1.12) with an Al_2O_3 content of 12.33% and a more acidic slag (C/S = 0.96) with an Al_2O_3 content of 7.36%.

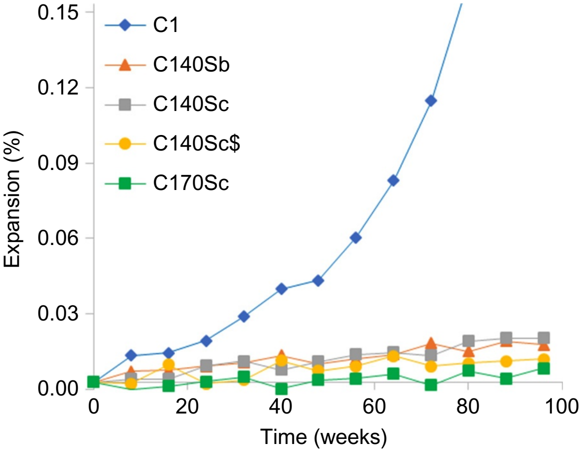

Fig. 17.8 Linear expansion of mortar prisms immersed in 3 g L^{-1} of Na_2SO_4 (Whittaker, 2014).

17.1.4.4 Other deterioration mechanisms

The use of GGBFS generally improves resistance to alkali–aggregate reactions. This is due to a number of reasons. Firstly, the reduced clinker content reduces the pH of the pore solution. Incorporation of aluminates into the C-S-H aids immobilisation of alkalis, whereas the presence of glassy, silicate-rich slag also alters the alkali-silica ratio. These factors all help to reduce the incidence of alkali–aggregate reactions. The reduced permeability of slag-cement composites also aids resistance to alkali–aggregate reactions.

Finally, the use of GGBFS in composite cements appears to have very little impact on the freeze–thaw resistance of concrete.

17.2 Natural pozzolans

The term ‘pozzolana’ has two distinct meanings. In its strictest definition, the term refers to pyroclastic (volcanic) ashes, found primarily near the town of Pozzuoli. More commonly, the term is used to describe inorganic materials, often siliceous or siliceous and aluminous, which show no hydraulic behaviour by themselves, but in the presence of moisture and a suitable alkaline activator show hydraulic behaviour to produce a cementitious material. This section adopts the broader definition to include materials of non-volcanic origin.

Pozzolana has been used for millennia, with the first reported use being about 5000 BC. However, it was the Romans who started to use them widely. Vitruvius mentions the pozzolana of Pozzuoli as being of the finest quality and as being used for the construction of the Roman port of Cosa despite being 2000 km from Pozzuoli. Pouring a mixture of lime, pozzolana and water into a wooden formwork which were then lowered into the sea, the Romans created a material of which remnants can still be seen today, more than 2100 years later.

The basis of this longevity is the pozzolanic reaction. This is a simple acid–base reaction between portlandite (or any other alkaline activator) and silicic acid (H_4SiO_4), as shown here:

$$\mathrm{Ca(OH)_2 + H_4SiO_4 \rightarrow Ca^{2+} + H_2SiO_4{}^{2-} + 2H_2O \rightarrow CaH_2SiO_4 \cdot 2H_2O}$$

Or it can be schematically done by using standard cement chemistry notation, as shown here:

$$\mathrm{CH + SH \rightarrow C\text{-}S\text{-}H}$$

17.2.1 Origin of natural pozzolans, composition and physical properties

Given the huge variety of natural pozzolans, it is somewhat difficult to make generalised statements about origin, composition and properties. Similarly, it is difficult even to attempt to classify them. Indeed, often the only property these materials have in common is their propensity to react with water and an alkaline activator to produce a cementitious binder.

Perhaps the first attempt at classification has already been imposed, by EN197-1, which allows the addition of either natural or artificial pozzolans (the latter including fly ashes, burnt shale and silica fume). A common classification of pozzolans is shown in Fig. 17.9. Considering natural pozzolans, the first division is between pyroclastic and clastic rocks. The former is produced from explosive volcanic eruptions where magma erupts into the air, with a rapid pressure decrease releasing gases which had been trapped in the magma. The molten rock then cools rapidly to yield a glassy material with an open pore structure.

These materials are then deposited as loose, heterogeneous ashes towards the base of the volcano and are classified incoherent materials. Examples of such pozzolana include those with earth from the Pozzuoli and Santorin areas. More common are coherent materials formed when the ashes are deposited and compacted before undergoing various weathering processes. Such weathering processes include zeolitisation, which improves pozzolanicity, or argillation, which diminishes it. Such materials include trass, such as Rheinisch trass and tuff.

Non-volcanic, or clastic pozzolana, also exist. These sedimentary rocks include clays and diatomaceous earths. The latter form from the siliceous skeletons of marine organisms known as diatoms. The former arise from the alteration of igneous rocks and may react with lime to produce C-S-H. However, in many instances, these materials are not suitable for replacing high levels of cement because of their high specific

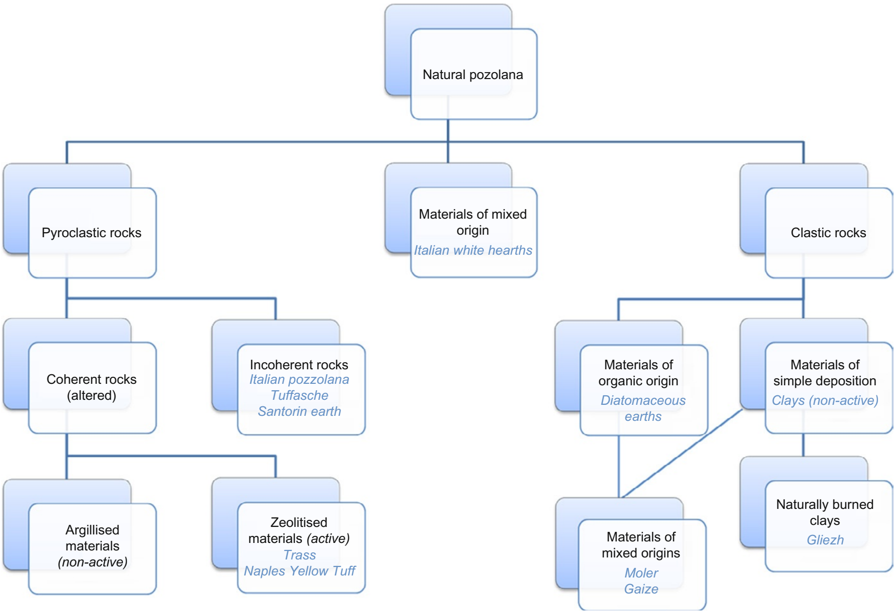

Fig. 17.9 Classification of pozzolans (cited by Hewlett, 2003).
After Massazza, F., 1976. Chemistry of pozzolanic additions and mixed cements. Il Cemento 1, 3–38.

surface area which leads to a higher water demand, which is detrimental to strength development.

Given the huge variation in origin, it is not surprising that pozzolana show huge variations in physical properties, plus chemical and mineralogical composition. Pozzolana are often ground to a specific surface area of 300–400 cm^2/g according to the Blaine method, that is, a similar fineness to Portland cement. However, the porous nature of some pozzolana can result in a higher water demand when used in concrete.

Chemical compositions can vary, but most pozzolana are rich in silica and alumina. Pyroclastic pozzolana then contain lower levels of lime, magnesia and iron oxides. Alkali contents can reach up to 10%, as can loss on ignition. Tuffs are also rich in silica and alumina, but show slightly more variable composition amongst minor elements. Their losses on ignition are also typically higher than for pyroclastic pozzolana. The composition of diatomaceous earths depends upon their clay contents. Those rich in clay often show higher alumina contents, whereas clay-poor examples are silica-rich. Because of their high silica and alumina contents, diatomaceous earths often contain very low levels of other elements. The elemental compositions of a number of pozzolana are shown in Table 17.5.

The mineralogy of pozzolana is yet more diverse than the elemental composition. However, the key phases to induce pozzolanic behaviour are glasses. Other phases present include zeolites (particularly in tuffs), quartz, feldspars and clay minerals. Diatomaceous earths may contain 25–100% opal, with the remainder being clay minerals, feldspars and quartz.

17.2.2 Pozzolanic reactions

A pozzolanic material may be defined by its ability to react with calcium hydroxide. Therefore, pozzolanic activity can be assessed by the extent and rate of the pozzolanic reaction. Methods for assessing pozzolanic behaviour may be direct or indirect, with the former measuring the consumption of calcium hydroxide and the latter measuring strength development.

Two common direct methods include the Frattini test (BS EN 196-5, 2011) and the saturated lime test (Thorstensen and Fidjestol, 2015; de Luxán et al., 1989). In the Frattini test, the material under investigation is mixed with Portland cement, whereas the saturated lime test involves placing the suspected pozzolan in a saturated calcium hydroxide solution. For both tests, activity is assessed by monitoring the consumption of calcium hydroxide, using a range of approaches, eg, XRD, thermal analysis or chemical titration. However, results from the two methods do not necessarily agree with one another (Donatello et al., 2010).

One of the most common indirect methods is the strength activity index (SAI). This method is performed by replacing a standardised proportion of Portland cement with the suspected pozzolan and comparing strength development against an equivalent 100% Portland cement mix. The SAI is then defined as the strength of the pozzolanic mix over the strength of the cement mix. Unsurprisingly, given that both tests blend the suspected pozzolan with Portland cement, results from the Frattini test correlate well with those from the indirect SAI test ($R^2=0.86$) (Donatello et al., 2010).

Table 17.5 **Elemental composition of a number of pozzolana (Massazza, 1976)**

Pozzolana	Origin	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	TiO_2	Na_2O	K_2O	SO_3	LOI
Incoherent pozzolana											
Bacoli	Italy	53.1	17.9	4.3	9.1	1.2	0.3	3.1	7.6	0.7	3.1
Vizzini	Italy	50.5	16.8	11.6	4.9	5.2	0.1	0.9	0.2	0.2	9.7
Santorin earth	Greece	65.1	14.5	5.5	3.0	1.1					3.5
Rheinische Tuffasche	Germany	58.9	19.5[a]	2.9	2.5	1.3		6.1	4.5		2.2
Pozzolanic tuffs											
Rheinische trass	Germany	52.2	18.3	5.8	4.9	1.2		1.5	5.1		11.1
Yellow tuff	Italy	54.7	17.7	3.8	3.7	1.0		3.4	6.4		9.1
Zeolite (clinoptilolite)	Japan	71.7	11.8	0.8	0.9	0.5		1.8	3.4	0.3	9.0
Clastic pozzolana											
Moler	Denmark	75.6	8.6	6.7	1.1	1.3		0.4	1.4	1.4	2.2
Diatomite	USA	86.0	2.3	5.8	Trace	0.6		0.2	0.2		8.3
Diatomite	USA	60.0	16.3	5.8	1.9	2.3					11.9
Gaize	France	79.6	7.1	3.2	2.4	1.0				0.9	5.9

[a] $Al_2O_3 + TiO_2$.

There are many factors affecting the reactivity of pozzolans, including particle size and fineness. The nature of the pozzolanic phase and its composition also has a big influence. Being that the glassy or zeolitic phases induce pozzolanic behaviour, the presence of other crystalline phases can diminish reactivity. Zeolitic phases are typically more reactive than glassy ones. Reactivity can sometimes be improved by gentle heating (to approximately 700–800°C), but heating to a temperature that is too high can lead to devitrification, which reduces reactivity.

As just mentioned, the pozzolanic reaction is that between calcium hydroxide and silicic acid to produce C-S-H. Therefore, replacement of cement clinker with pozzolana can induce the pozzolanic reaction once the clinker starts to produce portlandite. Given the prerequisite presence of portlandite, hydration of the pozzolana doesn't begin until after a few days, once about 80% of the alite has hydrated. Hydration of the cement–pozzolana blend is generally more gradual than that of a pure cement, although the filler effect can lead to an ever-so-slight acceleration of early age hydration. The pozzolanic reaction also leads to a lower heat evolution during hydration and a reduction in the portlandite content, with increased production of C-S-H (with implications for durability). Otherwise, the hydration products of a cement–pozzolana blend are similar to those from the hydration of pure Portland cement. There is a slight reduction in the calcium/silicon ratio of the C-S-H, whereas pozzolana high in alumina may lead to aluminium incorporation into C-S-H and the increased formation of alumina-rich phases such as ettringite, monosulphate and gehlenite (C_2ASH_8).

17.2.3 Effect of pozzolana on concrete performance

Blending pozzolana with cement can affect the early and late-age properties of concrete. The precise effects are very dependent upon the nature of the pozzolan. Pozzolana generally improve paste cohesiveness, thus reducing segregation. This may be ascribed to the residual negative charge on the surfaces of aluminosilicate pozzolana leading to their better dispersion. This can also help to explain the improved workability seen in cement–pozzolan blends. However, a greater factor here is particle shape. Smooth, round particles are known to improve workability, whereas rough, irregular particles diminish it. Particle fineness also affects workability, with finer particles leading to a stiffer paste. This is a particular problem for diatomaceous earths, where superplasticizers can be necessary to achieve suitable workability.

Setting times are a complex function of many factors, including cement and pozzolan fineness, pozzolan type, alkali content, replacement level, temperature and water content. As such, it is difficult to generalise about the effect of pozzolana on setting times. However, pozzolana typically extend setting times, particularly final setting times, especially at higher replacement levels. This may be explained by the reduced rate of the pozzolanic reaction.

Perhaps the most noticeable impact of blending pozzolana with cement is on strength development. The pozzolanic reaction consumes the non-binding portlandite to produce the cementitious binder C-S-H. The rate of this reaction is slower than that of cement hydration. Furthermore, portlandite needs to be produced in order to initiate the pozzolanic reaction. Therefore, early age strength development is compromised, but later-age performance can be improved. Massazza (1976) showed that at 10% replacement, the presence of a pozzolana had little effect; the filler effect compensates for any loss in early age performance. Higher replacement levels led to diminished early age performance, but led to greater strength beyond 3 months. Replacement levels over 30% led to diminished performance irrespective of the curing duration. At such replacement levels, insufficient portlandite is produced by the hydration of the cement to react with all of the pozzolana. Thus the remaining pozzolan acts as an inert filler, reducing the quantity of binder and so impacting strength.

Just as with strength, the effect of pozzolana on the porosity and permeability varies with the replacement level and curing duration. Early age porosity may be higher because of the slow pozzolanic reaction. Continued hydration, however, leads eventually to a lower porosity and therefore a lower permeability. As with GGBFS, replacement of cement with pozzolana can change the morphology of the C-S-H from fibrillar to foil-like, further reducing permeability.

17.2.4 Durability of pozzolanic cements

As with cement–GGBFS blends, provided that there has been adequate curing, the long-term reduction in permeability of cement–pozzolan blends helps to prevent the ingress of aggressive species into a cement paste and, therefore, a concrete, which can be beneficial for durability.

17.2.4.1 Resistance to carbonation

Two factors compete to affect the resistance of cement–pozzolana blends towards carbonation. Firstly, the pozzolanic reaction consumes portlandite, thus reducing the buffering capacity of the cement paste and so reducing resistance towards carbonation. Conversely, the pozzolanic reaction also reduces the permeability of paste, thus restricting the ingress of carbon dioxide and so protecting against carbonation.

At low levels of replacement, say 10%, the latter effect is predominant, so carbonation resistance is not affected, or may even be improved. At higher replacement levels cement–pozzolana blends are more susceptible to carbonation. However, it has been found that cement–pozzolana blends of equivalent strengths show comparable resistance to carbonation.

Given the slower rate of strength development, proper curing of cement–pozzolana blends is vital. Improperly cured samples have higher porosities and, because of a combination of clinker dilution and consumption through pozzolanic reactions, lower portlandite contents. These factors are both detrimental to carbonation resistance.

As with cement–GGBFS blends, the carbonation resistance of cement–pozzolana blends is significantly worse under accelerated carbonation conditions. This may be due to the coarsening of the microstructure seen under such conditions.

17.2.4.2 Resistance to chloride ingress

The refined pore structure of cement–pozzolana blends can help to prevent the ingress of chloride ions, thus protecting reinforcement steel from corrosion in reinforced concrete. Furthermore, the aluminate content of many pozzolana can help to increase the chloride binding capacity of the hardened cement paste, despite reduced C_3A contents because of clinker dilution. The chloride binding capacity is further increased by the increased C-S-H content brought about by the pozzolanic reaction.

A final benefit of cement–pozzolana blends exposed to chloride solutions is the reduced portlandite content. Cement samples exposed to sodium chloride solutions show increased evidence of portlandite leaching. The pozzolanic reaction reduces the availability of portlandite, thus reducing calcium leaching and improving durability.

17.2.4.3 Resistance to sulphate attack

Much as with cement–GGBFS blends, cement–pozzolana blends can show increased resistance to sulphate attack. The reduced portlandite contents plus the reduced C_3A contents because of clinker dilution both combine to improve the sulphate resistance of cement–pozzolana blends. These effects are in addition to the potential for reduced permeability in such systems.

In a study investigating the performance of blends containing Santorin earth, Mehta showed that increasing replacement of cement led to a reduced expansion of mortar prisms when immersed in a 10% sodium sulphate solution (Fig. 17.10) (Mehta, 1981).

17.2.4.4 Other deterioration mechanisms

As mentioned previously, some natural pozzolana contain inherently high levels of alkalis, (see Table 17.5). These materials may well be susceptible to alkali–aggregate reactions. Conversely, low-alkali pozzolana may actually help to resist expansive alkali–aggregate reactions since a lowering of the calcium/silicon ratio and incorporation of

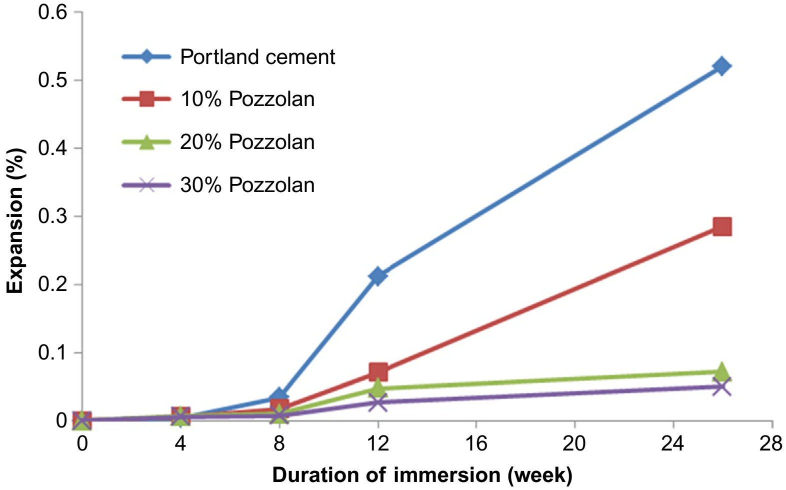

Fig. 17.10 Linear expansion of 28 days 2:1 mortar prisms exposed to 10% Na_2SO_4 solution. After Mehta, P.K., 1981. Studies on blended Portland cements containing Santorin earth. Cem. Concr. Res. 11 (4), 507–518.

aluminium into C-S-H both enable greater incorporation of alkalis into this phase. Perhaps, even more than with other systems, it is important to test the susceptibility of cement–pozzolan blends to alkali–aggregate reactions prior to use.

As with slag blends, there is no suggestion that cement–pozzolana blends have a detrimental impact on freeze–thaw resistance.

17.3 Fly ashes

17.3.1 Origin of fly ash, composition and physical properties

Fly ash, also known as pulverised fuel ash or PFA in the United Kingdom, is a by-product of the combustion of pulverised coal in thermal power plants. The ash is composed of the fine particles that are driven out of the boiler with the flue gases, before being collected by electrostatic precipitators or other particle filtration equipment. Meanwhile, ash which falls to the bottom of the boiler is called bottom ash. Fly ash typically comprises about 85% of the ash, with bottom ash accounting for the remaining 15%. Global production of coal combustion products stood at 780 Mt in 2011 (Heidrich et al., 2013), equating to about 660 Mt of fly ash.

The fly ash particles are formed upon the solidification of molten material suspended in the exhaust gases upon coal combustion. The rapid cooling of these particles leads to fly ashes generally being spherical, ranging in size from 0.5 to 300 μm. Furthermore, the rapid cooling of the ash prevents crystallisation of mineral phases, resulting in fly ash consisting primarily of a glassy aluminosilicate matrix. Nevertheless, crystalline phases may sometimes be observed. Fig. 17.11 shows an SEM micrograph of a typical PFA sample. It is not uncommon for fly ash to consist of hollow spheres, known as cenospheres. Consequently, the specific gravity of fly ashes, while variable, is lower

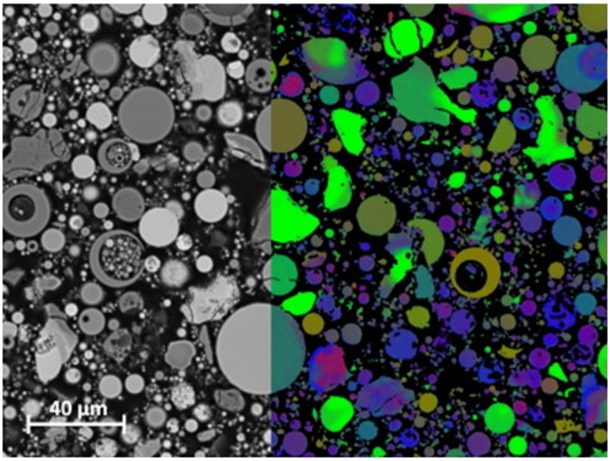

Fig. 17.11 BSE grey-level image of polished cross-sectioned PFA embedded in epoxy resin, showing the complex fly ash microstructure; an EDS overlay displays heterogeneity in distribution of elements (Durdziński et al., 2015).

than that of Portland cement, ranging from 1.9 to 2.9, but typically about 2.5 for Class F and slightly lower for Class C.

The composition of the incombustible components within the coal defines the composition of the fly ash, but all fly ashes include substantial amounts of silica (SiO_2), alumina (Al_2O_3) and lime (CaO). Generally, ashes from the combustion of subbituminous, or lignitic, coal gives an ash with a higher calcium content than bituminous (anthracite) coal. The majority of coal burnt in the United Kingdom is bituminous, so the relevant standard, EN450, just stipulates a lime content below 10%. However, in the United States subbituminous coal is more common. Consequently, ASTM C618 categorises fly ash as Class C when the lime content is greater than 20% and Class F when the lime content is less than 20%. Table 17.6 shows typical ranges in elemental composition for fly ashes arising from different coal types, and Table 17.7 shows the variation in composition of a number of flay ashes of different origin.

With their lower lime contents, Class F ashes from bituminous coals also show slightly lower magnesia and alkali contents, but higher iron contents. Similarly, their loss on ignition values are typically higher than Class C ashes. The ashes are, as mentioned previously, primarily glassy. However ashes may also contain some trace crystalline components, including quartz, mullite, magnetite and hematite. Being richer in calcium, Class C ashes may also contain some calcium-bearing phases, such as anorthite (CAS_2), gehlenite (C_2AS), akermanite (C_2MS_2) and various calcium silicates and calcium aluminates.

17.3.2 Fly ash hydration reactions

Class C fly ashes, with their moderate calcium contents, are latently hydraulic. That is, upon mixing with water, the ashes hydrate, albeit very slowly, to yield hydration products similar to those formed upon hydration of Portland cement, that is, ettringite, AFm and C-S-H.

Class F ashes meanwhile are pozzolanic and so react with the portlandite formed upon Portland cement hydration. Consumption of the portlandite therefore leads to lower levels of this phase within the hardened cement paste. Otherwise, the hydration of the cement–fly ash blend leads to the formation of the same hydration products as Portland cement, that is, ettringite, AFm and C-S-H. In addition, traces of strätlingite (C_2ASH_8), C_4AH_{13} and eventually hydrogarnet (C_3AH_6) may be formed. As with

Table 17.6 **Compositional variation of fly ashes from different types of coal**

Component	Bituminous	Subbituminous	Lignite
SiO_2 (%)	20–60	40–60	15–45
Al_2O_3 (%)	5–35	20–30	20–25
Fe_2O_3 (%)	10–40	4–10	4–15
CaO (%)	1–12	5–30	15–40
LOI (%)	0–15	0–3	0–5

Table 17.7 Composition of a range of fly ashes of different origins (Carette and Malhotra, 1987)

Type of coal	Chemical composition (wt%)												
	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	TiO_2	P_2O_5	MnO	BaO	SO_3	LOI
B	47.1	23.0	20.4	1.21	1.17	0.54	3.16	0.85	0.16	0.78	0.07	0.67	2.88
B	44.1	21.4	26.8	1.95	0.99	0.56	2.32	0.8	0.25	0.12	0.07	0.96	0.70
B	35.5	12.5	44.7	1.89	0.63	0.10	1.75	0.56	0.59	0.12	0.04	0.75	0.75
B	38.3	12.8	39.7	4.49	0.43	0.14	1.54	0.59	1.54	0.20	0.04	1.34	0.88
B	45.1	22.2	15.7	3.77	0.91	0.58	1.52	0.98	0.32	0.32	0.12	1.40	9.72
B	48.0	21.5	10.6	6.72	0.96	0.56	0.86	0.91	0.26	0.36	0.21	0.52	6.89
SB	55.7	20.4	4.61	10.7	1.53	4.65	1.00	0.43	0.41	0.50	0.75	0.38	0.44
SB	55.6	23.1	3.48	12.3	1.21	1.67	0.50	0.64	0.13	0.56	0.47	0.30	0.29
SB	62.1	21.4	2.99	11.0	1.76	0.30	0.72	0.65	0.10	0.69	0.33	0.16	0.70
L	46.3	22.1	3.10	13.3	3.11	7.30	0.78	0.78	0.44	0.13	1.18	0.80	0.65
L	44.5	21.1	3.38	12.9	3.10	6.25	0.80	0.94	0.66	0.17	1.22	7.81	0.82

B, Bituminous; SB, Subbituminous; L, Lignitic.

cement–slag blends, the nature of the C-S-H is modified; showing a lower C/S ratio, incorporation of some aluminium and a change in morphology from fibrillar to foil-like. Fig. 17.12 shows an electron micrograph of a 50% cement–fly ash blend hydrated for 180 days at 7°C. The figure shows a dense matrix of C-S-H, some portlandite and partially reacted fly ash particles.

Hydration of cement–fly ash blends is typically slower than that of pure Portland cement because of the nature of the pozzolanic reaction. However, the filler effect can accelerate early stage hydration of the clinker component and, as Fig. 17.12 shows, long-term hydration results in a dense, well-hydrated paste.

The reactivity of fly ashes depends on a number of factors, including chemical and mineralogical composition, fineness, morphology and glass content. No one single factor can be used to predict reactivity with accuracy, instead a number of factors combine to affect performance.

Numerous standards define the composition of fly ashes. Typically, the sum of SiO_2, Al_2O_3 and Fe_2O_3 must exceed a given threshold; BS EN450 stipulates a threshold of 75%, whereas ASTM C 618 stipulates ≥70% for Class F ashes and ≥50% for Class C ashes. The silica and alumina contents of fly ashes are known to correlate with long-term performance, although it is only the glassy component which shows pozzolanic behaviour. Silica and alumina bound in mullite or quartz will have a deleterious effect on pozzolanicity.

Of all the preceding factors, reactivity has been shown to correlate most strongly with particle fineness. Finer particles, with greater specific surface areas, react fastest. Furthermore, coarse particles (>45 μm) have a considerably negative effect on performance. This is primarily because these particles have the highest unburnt carbon content, thus greatly affecting the water demand.

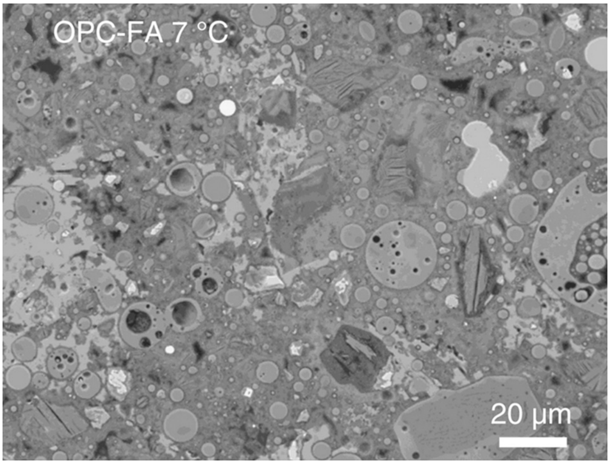

Fig. 17.12 SEM micrograph of a 50:50 cement–PFA blend after hydration for 180 days at 7°C (Deschner et al., 2013).

17.3.3 Effect of fly ash on concrete performance

As with natural pozzolana, fly ash affects the early and late-age properties of concrete, including cohesiveness, workability, setting time and strength development. However, again as with natural pozzolana, the precise effects depend on the fly ash in question. Factors such as chemical and mineral composition, carbon content, particle fineness and glass content can all affect performance.

Typically, cohesiveness is improved by the addition of fly ash. This is, in part, due to the slight negative charge on the surface of aluminosilicate minerals maintaining dispersion of the ash particles in the plastic cement paste. This effect partially explains the commonly observed increase in workability of cement–fly ash blends. A greater factor in the improved workability is the spherical nature of the fly ash particles, effectively acting as a lubricant. It has been estimated that a 10% replacement of cement with fly ash has an equivalent effect on compaction as increasing the water content by 3–4%. However, some very fine ashes or ashes with particularly high unburned carbon contents show decreased workability. Similarly, the composition of the fly ash can dramatically affect setting times and it is generally accepted that Class F ashes delay setting.

Another considerable benefit arising from the use of fly ash in concrete is a marked reduction in heat generation, with early age heat generation estimated to be 15–30% of that of Portland cement. Indeed, this has proven to be particularly beneficial for mass concrete, and the first application of fly ash concrete in the United Kingdom was for such purposes during the construction of the Lednock, Clatworthy and Lubreoch dams in the 1950s.

The presence of fly ashes in a concrete mix also has an effect on strength development. Again, this depends on the nature of the fly ash. Class C ashes, being self-cementing, may have no impact on strength development. Class F ashes meanwhile are not self-cementing, so they contribute to long-term strength via the pozzolanic reaction. This reaction, being slower than the hydration reactions of cement clinker, leads to lower early age strengths but higher later-age strengths. However, extended periods of moist curing are essential for cement–fly ash blends to achieve their full potential. Attempts to relate strength development to composition or fineness, however, have proven fruitless, there seemingly being too many factors at play to accurately predict performance.

The pozzolanic reaction, in addition to contributing to strength, reduces porosity and permeability. However, the reduced permeability of cement–fly ash blends is seen only after prolonged periods of curing. At early ages, and even after curing for 28 days, the permeability of cement–fly ash blends can be greater than that of an equivalent pure cement mix. In order to attain low permeability, it is essential for cement–fly ash blends to undergo prolonged moist curing.

17.3.4 Durability of cement–fly ash blends

As previously alluded to, the durability of cement–fly ash blends is greatly influenced by the degree of hydration. Prolonged periods of moist curing, essential for a continued pozzolanic reaction, greatly improve the durability of cement–fly ash blends. However, the pozzolanic reaction also consumes portlandite, with potential implications for resistance towards carbonation.

17.3.4.1 Resistance to carbonation

As with the systems mentioned earlier in this chapter, the consumption of portlandite in the pozzolanic reaction to produce C-S-H, with an associated reduction in porosity and permeability is somewhat of a double-edged sword with regards to carbonation resistance. A number of studies have shown that the carbonation resistance of cement–fly ash blends is less than that of pure cement blends. Conversely, studies have also shown that, provided the concretes were of equivalent strength, the replacement of cement with fly ash has no impact on carbonation resistance. What is absolutely clear from a great many studies is that cement–fly ash blends which are exposed to aggressive environments after just short periods of moist curing (eg, 1 day) are more prone to carbonation. Under such conditions, the hardened cement pastes are more permeable, with reduced portlandite contents because of dilution of the cement and consumption of the portlandite during the pozzolanic reaction. This effect is more pronounced in lower-strength concretes, where permeability is greater.

The apparent diminished performance of cement–fly ash blends in laboratory-based accelerated carbonation studies is not necessarily matched by field studies. As with cement–GGBFS blends, fly ash composite cements show equivalent performance as pure cement systems during field exposure tests. This may be explained by the finer pore structure becoming saturated at lower relative humidities such that brief periods of rain leave the concrete saturated longer, thus reducing the transport of carbon dioxide into the matrix.

17.3.4.2 Resistance to chloride ingress

As with cement–GGBFS blends, the finer pore structure of cement–fly ash blends reduces permeability and thus helps to resist chloride ion ingress. Furthermore, the high alumina content of many fly ashes increases chloride binding capacity such that chloride ions can be bound in Friedel's salt. The reduced rate of hydration compared to slag, however, means that, as with carbonation and strength, improved performance of cement–fly ash blends depends on prolonged curing (eg, see Khatib and Mangat, 2002).

17.3.4.3 Resistance to sulphate attack

The resistance of PFA–cement blends to sulphate attack is mixed, with some systems showing improved performance with the addition of PFA, and some showing diminished performance. Improvements in performance generally stem from the lower permeabilities brought on by the pozzolanic reaction. However, the increased aluminium and iron contents of the PFA can cause problems because of the involvement of these elements in the formation of ettringite. Because a source of calcium is required for the formation of ettringite, the calcium content of PFA appears important.

In an attempt to assess the ability of PFA to improve sulphate resistance, Dunstan (1980) proposed the use of a resistance factor (R), calculated as

$$R = (C - 5) / F$$

where C is the percentage of CaO, and F is the percentage of Fe_2O_3. The R value was then related to sulphate resistance, as shown in Table 17.8. Although there was some discussion as to the widespread applicability of the R value, a number of studies showed Class C ashes performed worse than Class F ashes, as previously explained by the availability of calcium to form ettringite.

17.3.4.4 Other deterioration mechanisms

At moderate levels of replacement, the replacement of cement with PFA has no effect on the freeze–thaw resistance of concrete. At higher levels (eg, 50%), the effect on freeze–thaw resistance is detrimental. However, such levels of replacement are far in excess of those where the portlandite produced from cement hydration is sufficient to induce complete reaction of the PFA via the pozzolanic reaction. Despite the minimal impact on freeze–thaw resistance at moderate replacement levels, it is worth noting that the use of air entraining admixtures are required for freeze–thaw resistance, and the addition of PFA to concrete increases the demand for such an admixture for effective distribution of air bubbles throughout the hardened cement matrix.

The impact of PFA additions on alkali–aggregate reactions is mixed. Again, huge variations in composition make generalisations difficult. However, high-calcium PFAs appear to offer less resistance. In part, this is likely because of the higher calcium/silicon ratio of the resultant C-S-H in such systems being less able to ‘mop up’ excess alkalis within the cement paste. Similarly, the high inherent alkali contents of some PFA samples can make such materials susceptible to alkali–aggregate reactions. With the provisos of low calcium and alkali contents, PFA blends can perform well with regard to alkali–aggregate reactions, with more pozzolanic ashes performing better. This may give a clue as to why replacement with PFA is not generally considered as effective as replacement with GGBFS, with the lower early age reactivity of PFA offering less potential for immobilisation of alkalis.

Table 17.8 **The relationship between *R*-value and sulphate resistance**

R-value[a]	Sulphate resistance[b]
<0.75	Greatly improved
0.75–1.5	Moderately improved
1.5–3.0	No significant change
>3.0	Reduced

[a] At 25% replacement.

[b] Relative to ASTM type II cement at a *w/c* ratio of 0.45.

After Dunstan, E.R., 1980. A possible method for identifying fly ashes that will improve the sulphate resistance of concretes. Cement Concrete Aggr. 2, 20–30.

17.4 Silica fume

17.4.1 Origin of silica fume, composition and physical properties

Silica fume, also known as microsilica, is an extremely fine noncrystalline polymorph of silica and is produced in electric arc furnaces (EAFs) as a by-product of silicon and ferrosilicon alloy production. While the first trials of silica fume as an additive in concrete were in the 1950s, at that time much of the silica fume was simply discharged into the atmosphere. It was only with increased environmental legislation in the late 1960s that silica fume began to be collected in bag filters, whereupon manufacturers began to search for a use for the material. With the preliminary studies in Norway having shown silica fume's potential for improving the durability of concrete, a potential market was made.

As just mentioned, silica fume arises as a by-product of silicon or ferrosilicon alloy formation in an EAF. In this process, quartz is heated to about 2000°C with the addition of a source of carbon to reduce the quartz to silicon. Small quantities of silicon vapour are released from the furnace whereupon it oxidises and condenses to form microspheres, which are then collected via fans and bag filters.

As produced, the silica fume has a bulk density of about 200–300 kg/m^3, which is impractical for use in concrete. However, densified silica fume has a bulk density of about 500 kg/m^3 and may find uses. Increasingly, with concerns over silicosis, silica fume is provided to the concrete industry in slurry form, with a specific gravity of about 1400 kg/m^3. The use of a slurry has been shown to enable better dispersion of the silica fume within the concrete mix, leading to a more homogeneous material and better quality control.

Silica fume comprises tiny microspheres, as shown in the scanning transmission electron micrograph in Fig. 17.13 (Mitchell et al., 1998). These range in diameter from about 20 to 250 nm, with an average diameter of about 150 nm. Fig. 17.14 shows

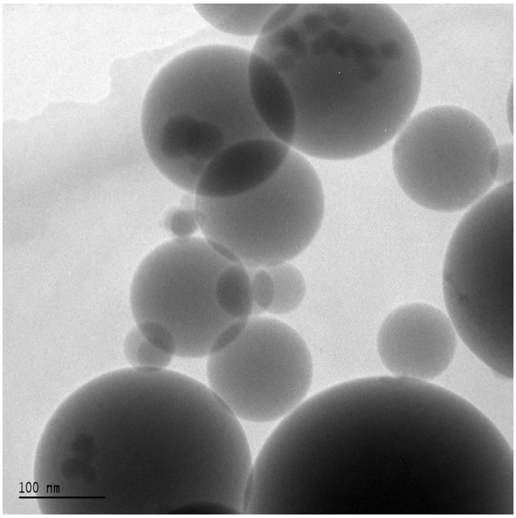

Fig. 17.13 Transmission of an electron micrograph of silica fume agglomerate (Mitchell et al., 1998).

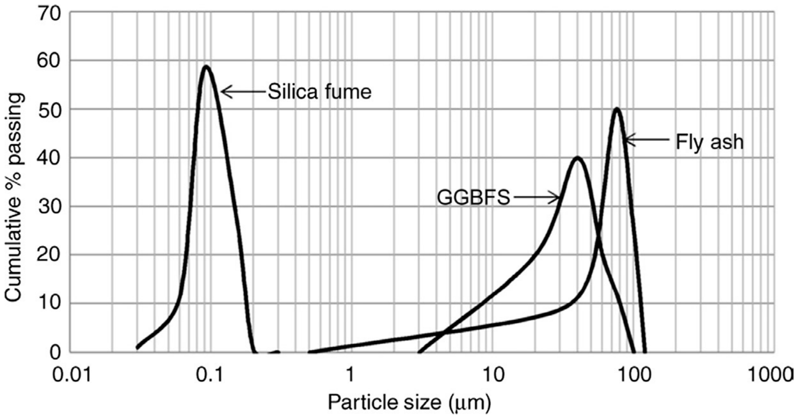

Fig. 17.14 Particle size distribution of silica fume, compared to a number of other components found within composite cements (Ince et al., 2015).

a typical particle size distribution for silica fume (Ince et al., 2015) where the much finer nature of the material is clearly evident. The specific surface area of silica fume is 13,000–30,000 m^2/kg.

Silica fume comprises primarily amorphous silica. The precise silica content depends on the origin of the silica fume, but it always exceeds 85%. Small quantities of crystalline silica exist, often as crystobalite. Other trace contaminants include, in approximately decreasing contents, carbon (arising from the addition of coke and wood chips during the production process), aluminium, iron, calcium, magnesium, potassium and sodium.

17.4.2 Silica fume hydration reactions

The very high specific surface area of silica fume, together with its primarily amorphous nature, means that silica fume is an extremely efficient pozzolan. Its addition to concrete has a number of effects, as explained here:

(i) As with other pozzolans mentioned earlier, the addition of silica fume affects cement hydration. The portlandite produced upon alite and belite hydration can react with the silica fume to form additional C-S-H, of a slightly lower calcium/silicon ratio than found in a CEM I cement. Furthermore, the fine particles provide nucleation sites on which the hydration products can form and increase the effective water-to-cement ratio during the early stages of hydration.

(ii) The very fine particles also modify the pore structure of the hardened cement by filling spaces between particles. This leads to a considerable reduction in the volume of large pores and densification of the cement matrix.

(iii) The consumption of portlandite modifies the interfacial transition zone (ITZ) between the cement paste and the aggregate. There is a slight reduction in ITZ thickness with the use of silica fume, plus a reduction in the extent of portlandite crystal orientation within the zone. These two factors combine to improve the mechanical properties and durability of silica fume concrete. (Incidentally, for this reason, the impact of silica fume is far greater on concrete than on silica fume–cement pastes.)

Being an extremely efficient pozzolan, typical replacement levels in silica fume concrete are lower than for other pozzolans. Replacement levels much greater than 15% appear to be inefficient in terms of pozzolanic behaviour, although silica fume will continue to provide a space-filling effect at higher replacement levels. Consequently, especially considering the higher price of silica fume compared to other pozzolans, replacement levels in practise are typically no more than 20%. At such levels there can be a slight increase in early age heat evolution, again likely attributed to the accelerating filler effect. Longer-term heat evolution is typically reduced.

17.4.3 Effect of silica fume on concrete performance

The addition of silica fume to concrete has a minimal effect on both initial and final setting times at replacement levels of 5–10%, but it leads to a slight increase in setting times at higher replacement levels. Again, this may be understood by considering the competitive effects of the filler effect and the pozzolanic reaction. At low replacement levels, the two effects are roughly equal in impact, whereas at higher replacement levels, the latter effect dominates.

The fine nature of silica fume and the resultant incredibly high specific surface area can lead to a reduction in concrete workability. However, workability is often regained upon vibration of the concrete thanks to the lubricating effects of the spherical silica fume particles. As such, the slump test may not be an effective measure of concrete containing silica fume. Furthermore, as a consequence of the reduced workability, it is often necessary to use a water-reducing admixture in such concrete. The high specific surface area also brings benefits, in the form of improved cohesion and reduced bleeding.

The use of silica fume in concrete increases its compressive strength. Replacement levels of 10% can readily increase compressive strengths by 30–50%. As such, the use of silica fume is imperative for the production of high-strength concrete. However, the high water demand of silica fume concrete necessitates the use of a water-reducing admixture. Furthermore, as with many other SCMs, prolonged moist curing is essential for concrete containing silica fume.

17.4.4 Durability of cement–silica fume blends

17.4.4.1 Resistance to carbonation

As with PFA and other pozzolans, the effect of silica fume on carbonation resistance is not clear-cut. Consumption of portlandite during the pozzolanic reaction will decrease the quantity of carbonateable matter within the cement paste. However, the reaction will also reduce the capillary porosity and hence reduce the ingress of carbon dioxide into the matrix. As such, comparing the impact of silica fume use on carbonation resistance depends on a great many factors, such as replacement level, the water/binder ratio, curing and exposure conditions.

Recent work has shown that at low water/binder ratios, the replacement of cement with silica fume had minimal impact on the carbonation of concrete, whereas higher water/binder ratios (≥0.60) had a detrimental impact (Kulakowski et al., 2009).

17.4.4.2 Resistance to chloride ingress

The refined pore structure induced in concrete containing silica fume as a result of the pozzolanic reaction and densification of the cement matrix leads to a marked reduction in permeability. Consequently, replacement of cement with silica fume can greatly reduce the ingress of chlorides into concrete (Khatib and Mangat, 2002). Furthermore, since the majority of chlorides are bound via adsorption onto the C-S-H, the additional C-S-H formed as a result of the pozzolanic reaction provides an additional chloride-binding capacity within the system. Fig. 17.15 shows the chloride profile obtained from mortar samples in which silica fume had replaced either the cement or the aggregate (Papadakis, 2000). The reduction in chloride penetration is clearly evident.

17.4.4.3 Resistance to sulphate attack

As with other deterioration mechanisms, the effects of the addition of silica fume on sulphate resistance are complicated. Different measures of the degree of sulphate attack can give contradictory information. Using standard mortar expansion data, the addition of silica fume may appear to improve sulphate resistance. In a study comparing the performance of mortars containing up to 15% silica fume exposed to either sodium or magnesium sulphate, the addition of silica fume showed improved sulphate resistance, with reduced mortar bar expansion (Fig. 17.16). Since silica fume is almost pure silica, its incorporation into concrete does not upset the aluminium:sulphate balance which is affected during sulphate attack. Therefore, the primary effect of the addition of silica fume was its pozzolanic nature leading to reduced porosity and, therefore reduced sulphate ingress. Consequently, while showing reduced expansion, the mortar bars also showed only minimal loss of strength after immersion for up to 9 months, with the silica fume blends slightly outperforming the pure cement mortar.

However, the situation upon immersion in a magnesium silicate solution was very different (Fig. 17.17). Magnesium sulphate attack, in addition to upsetting the

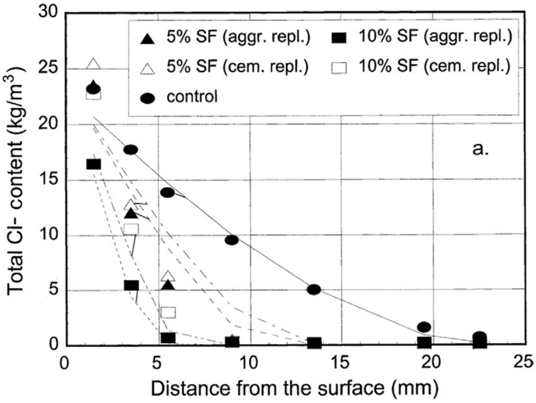

Fig. 17.15 Normal chloride penetration profiles for mortars incorporating silica fume (Papadakis, 2000).

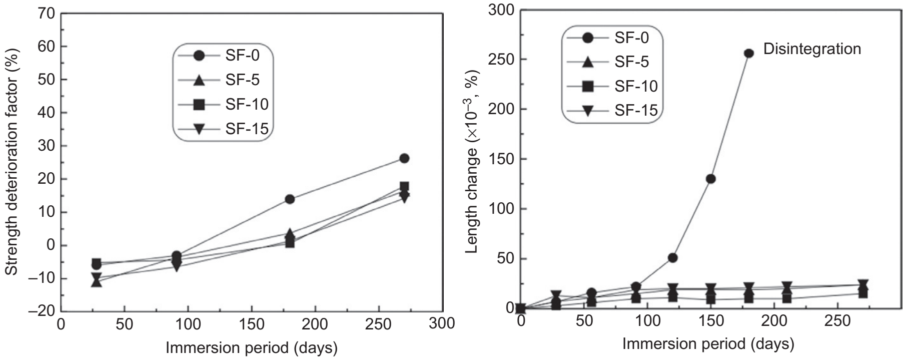

Fig. 17.16 Variation in strength deterioration factor (*left*) and mortar expansion (*right*) for mortar specimens with and without silica fume upon immersion in a 5% sodium sulphate solution (Moon et al., 2003).

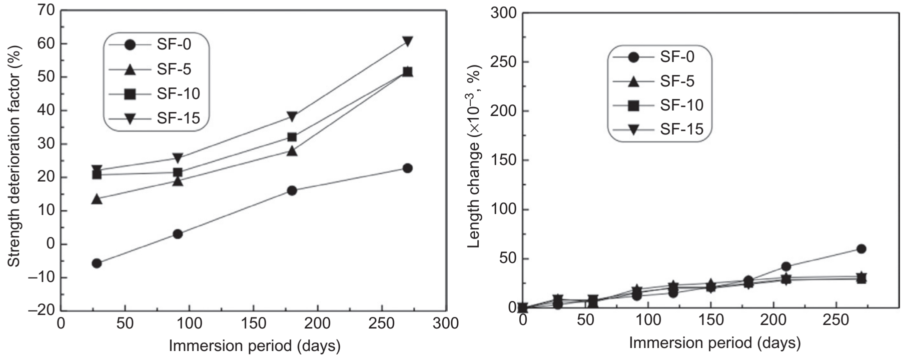

Fig. 17.17 Variation in strength deterioration factor (*left*) and mortar expansion (*right*) for mortar specimens with and without silica fume upon immersion in a 5% magnesium sulphate solution (Moon et al., 2003).

aluminate:sulphate balance involved in the formation of ettringite, can result in destruction of the C-S-H structure. Consequently, samples containing silica fume which were immersed in a 5% magnesium sulphate solution, while showing little evidence of expansion, did show a considerable loss of strength (Moon et al., 2003).

17.4.4.4 Other deterioration mechanisms

The high reactivity of silica fume makes it effective in combating alkali–silica reactions. The finely divided silica can react with and 'mop up' excess alkalis, thus reducing the pH of the pore solution, and so reduce the formation of any expansive alkali–silica reaction (ASR) gel.

The effects of silica fume addition on freeze–thaw resistance are mixed. Some authors have reported worse freeze–thaw resistance in samples containing silica fume, whereas others have shown that silica fume addition has either no effect or a slightly positive influence. What does appear clear is that a combination of silica fume and an air entraining admixture can offer improved freeze–thaw resistance, with the silica fume helping to lead to a better dispersion of air bubbles in the hardened concrete.

17.5 Metakaolin

17.5.1 Origin of metakaolin, composition and physical properties

Unlike many of the other SCMs mentioned so far, metakaolin is not an industrial by-product, but is a manufactured product produced under carefully controlled conditions. It is produced by heating kaolin, a natural clay mineral, to temperatures typically between 650°C and 750°C. At such temperatures, the kaolin is dehydroxylated, transforming from kaolin ($Al_2Si_2O_5(OH)_4$ or AS_2H_4) to metakaolin ($Al_2Si_2O_5$ or AS_2). This process results in a disordered structure and a highly reactive, amorphous pozzolanic material. Heating to temperatures higher than those mentioned leads to 'dead burned' material and the formation of nonreactive mullite.

Metakaolin often shows a platy morphology, with particle sizes of about 2 μm in diameter. It is often white in colour, so it is often preferred for high-quality architectural applications. While metakaolin is predominantly an aluminosilicate, being derived from natural minerals, it can show slight variations in chemical composition. The silica content can vary from about 50% to 55%, and the alumina content can vary from about 40% to 45%, while other phases present include Fe_2O_3, TiO_2, SO_3, P_2O_5, CaO, MgO, Na_2O and K_2O.

17.5.2 Metakaolin hydration reactions

The hydration reactions of metakaolin blends depend on the composition of the cement with which the metakaolin is blended. When blending with a CEM I pure Portland cement, the principle reaction is the pozzolanic reaction between the metakaolin and portlandite arising from cement hydration. This produces an aluminium-substituted C-S-H phase (C-A-S-H) plus some crystalline phases such as strätlingite (C_2ASH_8), C_4AH_{13} and hydrogarnet (C_3AH_6). Furthermore, in the presence of carbonates, calcium carboaluminates may form.

As with the other SCMs, the consumption of portlandite by metakaolin with the formation of additional cementing products can improve the performance of cement–metakaolin composites (Wild and Khatib, 1997).

17.5.3 Effect of metakaolin on performance of concrete

Because of its fine particular nature, metakaolin has a higher water demand yet better consistence than some other SCMs—for example, PFA. Snelson et al. (2011) reported a steady, almost linear increase in water demand in systems with up to 20% replacement. However, in the same study, they showed that the impact on setting times was a little more complicated. Replacement of cement with metakaolin leads to an increase in initial and final setting times, the extent of which is dependent on the level of replacement. For mortars of standard consistency, at 5% replacement, initial and final setting times increased by about 30 and 45 min, respectively, over a Portland cement mortar. Retardation was slightly less at 10% and 15% replacement, before increasing further at 20% replacement. These effects were explained by considering the different phases formed in the blends at different levels of replacement, in part because of the increased water demand with increasing replacement.

As with the other SCMs mentioned to date, the pozzolanic reaction between metakaolin and portlandite produces additional C-S-H, which contributes to an increase in strength. Being such an effective pozzolan, metakaolin is particularly effective at improving early age strength. Furthermore, as with silica fume, metakaolin densifies the ITZ (Wild et al., 1996).

17.5.4 Durability of cement–metakaolin blends

As with silica fume, the addition of metakaolin to concrete can improve performance over and above the increased compressive strength arising as a result of the pozzolanic reaction. The use of metakaolin improves the ITZ, which can inhibit the transport of aggressive species through the concrete, thus improving performance (Khatib and Wild, 1996).

17.5.4.1 Resistance to carbonation

The effects of the addition of metakaolin to concrete are twofold and contradictory. The pozzolanic reaction consumes portlandite, thus reducing resistance to carbonation. However, the pozzolanic reaction also reduces the capillary porosity, thus decreasing carbonation rate. The precise impact will depend upon the maturity of the concrete and the level of replacement. However, typically the former effect slightly dominates over the latter such that carbonation resistance is often slightly compromised when cement is blended with metakaolin.

17.5.4.2 Resistance to chloride ingress

Resistance to chloride attack depends on two factors: the ability for the chloride ions to penetrate the hardened cement paste and the ability of the hardened cement paste to bind the chlorides. The latter factor depends on the quantity of C-S-H within the hardened cement paste (which is responsible for about 65% of the chloride binding) and the aluminate content (which binds the chlorides as Friedel's salt).

The addition of metakaolin to concrete is beneficial in all of these regards. The pozzolanic reaction reduces permeability, while increasing the C-S-H content. Furthermore, the high alumina content of metakaolin provides a source of reactive alumina with which chlorides can be bound as Friedel's salt.

Poon et al. (2006) investigated the chloride penetrability of concretes blended with silica fume and metakaolin. Both systems significantly reduced chloride ingress, and while silica fume appeared more effective at 5% replacement, higher levels of metakaolin replacement were far more effective (Table 17.9).

Table 17.9 **Chloride penetrability of control and blended concretes (Poon et al., 2006)**

Series	w/b	Mix	Total charge passed (C)			
			3 days	7 days	28 days	90 days
1	0.3	Control	2461	2151	1035	931
		5% MK	1327	1244	862	646
		10% MK	417	347	199	135
		20% MK	406	395	240	124
		5% SF	1060	945	665	426
		10% SF	567	445	360	336
2	0.5	Control	5312	4054	2971	2789
		5% MK	4215	3765	2079	1065
		10% MK	1580	1247	918	752
		20% MK	751	740	640	580
		5% SF	3156	2047	1641	1235
		10% SF	3140	1877	1523	1053

17.5.4.3 Resistance to sulphate attack

As with GGBFS, the replacement of cement with metakaolin in concrete has been shown to have a clearly positive impact on sulphate resistance (Khatib and Wild, 1998). The pozzolanic reaction reduces portlandite content and refines the pore structure such that aggressive species cannot ingress into the concrete so readily. More importantly, metakaolin has a high aluminate content. The increased availability of aluminates might be expected to increase the likelihood of expansive ettringite formation. However, it appears that the excess of aluminates enables the formation of monosulphate rather than ettringite during exposure to sulphates.

17.5.4.4 Other deterioration mechanisms

The replacement of cement with metakaolin may dilute the alkali content of cement and lead to densification of the cement paste, both of which can improve concrete's resistance to the alkali–silica reaction. The use of metakaolin leads to the incorporation of aluminium into the C-S-H structure, which increases the phase's ability to bind alkalis, further reducing susceptibility to the alkali–silica reaction. Metakaolin has also been shown to improve or have no effect on the freeze–thaw resistance of concrete.

17.6 The environmental benefits of SCMs

The production of 1 tonne of Portland cement in Europe releases 884 kg of carbon dioxide. A little over half of this effect is associated with the calcination of limestone, with just over a third accounted for by the fuel used to attain the high temperatures required in the cement kiln. Just about 10% of the effect is attributed to either grinding operations or transportation costs around the plant (emissions from the plant to the site obviously fall outside of this analysis). Aggregates meanwhile have a much lower carbon footprint. Flower and Sanjayan (2007) reported values of 0.0459 tonne CO_2eq/tonne for granite/hornfels and 0.0357 tonne CO_2eq/tonne for basalt. Because of the lack of a crushing step, sand had a much lower carbon emission of 0.0139 tonne CO_2eq/tonne. Many people have now conducted life-cycle assessments of concrete, and while there may be slight variations in the precise carbon dioxide emission values for each component of concrete (depending on, eg, kiln type, fuel use within the kiln and grinding method), what remains true is that it is the emissions associated with cement which dominate the environmental footprint of concrete production.

Approximately 90% of carbon dioxide emissions associated with the manufacture of concrete are attributed to the cement. Although it is possible to reduce the carbon dioxide emissions with a carefully designed concrete mix (Purnell and Black, 2012; Damineli et al., 2010), perhaps one of the most effective means of reducing the carbon footprint of concrete is to replace cement with a binder having a lower carbon footprint. SCMs perform such a function.

Many SCMs are industrial by-products, burning coal in the case of PFA, for example. Since these operations would continue regardless of whether or not the SCMs were used, convention dictates that emissions associated with the primary product are assigned to the primary product. The emissions associated with SCMs are due to activities post-production, such as grinding, refining and transportation. Consequently, SCMs provide a clear opportunity to reduce emissions. Furthermore, the use of SCMs in concrete has additional benefits. The incorporation of SCMs in concrete offers a use for the materials, as opposed to sending them to a landfill. Secondly, with 1 tonne of Portland cement requiring 1.6 tonnes of limestone and clay for its manufacture, the replacement of cement by SCMs reduces abiotic depletion. Finally, as discussed previously, SCMs can improve the durability of concrete, thereby prolonging the lifetime of structures and reducing the environmental burden associated with repair and maintenance. This section considers the environmental benefits of each of the SCMs mentioned so far, focussing on carbon dioxide emissions. The key points are illustrated in Table 17.10.

17.6.1 Emissions associated with GGBFS

The carbon dioxide emissions associated with GGBFS production are due to the need for rapid cooling of the slag, its grinding and any transportation from the point of production to the grinding facility. Flower and Sanjayan (2007) used a figure of 0.143 tonne CO_2eq/tonne. Meanwhile King (2012) quotes 0.052 tonne CO_2eq/tonne, the UK Mineral Products Association (2015) quotes a value of 0.067 tonne CO_2eq/tonne, while Hammond and Jones (2008) report a value of 0.083 tonne CO_2eq/tonne. At replacement levels of up to 70%, GGBFS has the potential to reduce the embodied carbon of concrete by two thirds. More common replacement levels of 30–40% can still reduce the carbon footprint of concrete by 25–35%.

An often neglected environmental footprint is the water used in the production of a material, that is, virtual water. According to Hewlett (2003), the production of 1 tonne of GGBFS requires 3 tonnes of water when using conventional water jets for cooling,

Table 17.10 Summary data for various SCMs

SCM	**Typical (max.[a]) replacement level (%)**	**ECO$_2$ (tonne CO$_2$eq/tonne)**	**Impact on durability**	**Notes**	**Ref.**
GGBFS	40 (95)	0.052–0.143	Carbonation+ Chloride ingress+ Sulphate resistance+ Other +	The virtual water of GGBFS is greater than that of cement Reactivity of GGBFS is dependent on composition	Flower and Sanjayan (2007), King (2012), Hammond and Jones (2008), Mineral Products Association (2015)
Natural pozzolans	6–35 (35)	0.004–0.435	Carbonation± Chloride ingress+ Sulphate resistance+ Other±	The huge variability in these materials makes a value for embodied carbon very difficult to estimate. Lower values represent naturally pozzolanic materials while higher values are for thermally activated clays	Heath et al. (2014)
PFA	20 (35)	0.004–0.027	Carbonation− Chloride ingress+ Sulphate resistance± Other±	Prolonged moist curing is essential for any improved durability	Flower and Sanjayan (2007), King (2012), Hammond and Jones (2008)
Silica fume	5–10 (10)	0.014 (slurry) 0.028 (powder)	Carbonation± Chloride ingress+ Sulphate resistance± Other±	The CO$_2$ emissions include mixing, transfer and storage of silica fume slurry, from the production site in Norway to the UK	King (2012)
metakaolin	8–20	0.330–0.423	Carbonation− Chloride ingress+ Sulphate resistance+ Other+	Embodied carbon figures are much higher than the other SCMs mentioned because metakaolin is not an industrial by-product	Hammond and Jones (2008), Heath et al. (2014)

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[a] Maximum as defined by EN197-1.

or 1 tonne of water when using a semidry pelletising process. This compares with 0.01–0.11 m^3/tonne for aggregates, 0.012–0.6 m^3/tonne for cement (with the higher value representing figures from the wet production process which is becoming obsolete in the United Kingdom. Incidentally, the figure for steel is 285 m^3/tonne, meaning that the virtual water of reinforced concrete is dominated by the reinforcement (Highways Agency, 2010).

17.6.2 Emissions associated with natural pozzolans

Of all of the SCMs covered here, natural pozzolans are the most difficult one in regards to finding reliable information on their environmental benefits. This difficulty is because of the wide variety of natural pozzolans, with different degrees of preprocessing required. It is also because, of all the various SCMs in use, natural pozzolans are the least prevalent ones.

It has not been possible to find comprehensive data for the embodied carbon of various natural pozzolans. Heath et al. (2014) recently presented figures for metakaolin and also meta-betonite of 0.423 and 0.435 tonne CO_2eq/tonne, which seems a reasonable estimation for a range of natural pozzolans requiring thermal pre-treatment. For those materials where there is no such need, it should be possible to estimate embodied carbon based on reported values for other materials which undergo similar treatment. For example, some of the volcanic ashes, as used by the Romans, require no pretreatment, so one can assume that they have embodied carbon values similar to PFA, that is, 0.004–0.027 tonne CO_2eq/tonne. Those pozzolans which require grinding (eg, trass) may have embodied carbon values similar to other ground mineral species, such as limestone, for which King reported a value of 0.032 tonne CO_2eq/tonne for limestone fines.

It is worth noting that, by not being industrial by-products, natural pozzolans don't offer some of the other aforementioned environmental benefits. These are mined materials and, therefore, contribute to abiotic depletion. Similarly, they are not diverting material from landfills. However, by again drawing comparisons with the production of other mineral components, the virtual water content of natural pozzolans may be estimated as 0.01–0.11 m^3/tonne, that is, the value reported for aggregates (Highways Agency, 2010).

17.6.3 Emissions associated with fly ashes

The carbon dioxide emissions associated with fly ash are lower than those associated with GGBFS, which is due to the fact that they do not need to be grinded. As with other SCMs, some variability is reported in the embodied carbon values for fly ash. Flower and Sanjayan (2007) report a value of 0.027 tonne CO_2eq/tonne, whereas others have reported lower values. King (2012) and the UK Mineral Products Association (2015) both reported a value of 0.04 tonne CO_2eq/tonne, whereas Hammond and Jones (2008) reported a value of 0.008 tonne CO_2eq/tonne. The consistently higher values reported by Flower and Sanjayan appear to be due to their assumption of 100 km of transportation in their calculations.

Replacement levels for PFA are typically about 20%, with EN197-1 stipulating a maximum replacement level of 35%. Despite the negligible embodied carbon of PFA, the reduction in carbon dioxide emissions is about 80% of the replacement level. This is because the lower reactivity of PFA compared to cement necessitates a slightly increased cement content to maintain performance. While replacement levels greater than 35% may be possible when using Class C fly ashes, the impact on durability may be too great, so these would fall outside EN197-1.

Any benefits associated with improved longevity are project-dependent. However, to achieve improved durability, prolonged moist curing is essential with PFA blended cements.

17.6.4 Emissions associated with silica fume

Data on the embodied carbon of silica fume is hard to find. King (2012) reported a value of 0.014 tonne CO_2eq/tonne of slurry, which equates to 0.028 tonne CO_2eq/tonne for silica fume. This value includes production of the silica fume and slurry mixing in Norway plus transport to the United Kingdom. As with PFA, the virtual water of silica fume, while unreported, is unlikely to be great.

With replacement levels of 10% or less, the replacement of cement with silica fume will not achieve significant reductions in the embodied carbon of any concrete, 10% at best. However, the environmental benefits arise when considering the whole-life performance of a concrete structure. The replacement of cement with silica fume brings two possible environmental benefits. Firstly, durability is improved, so the environmental impacts of repair or maintenance are reduced. Secondly, as mentioned previously, the addition of silica fume to concrete can improve strength by 30–50%, facilitating the production of high-strength concrete. This, in turn, means less concrete is used. However, the potential improvements in environmental footprints become project-specific, thus are beyond the remit of this section.

17.6.5 Emissions associated with metakaolin

Metakaolin, as with natural pozzolans, differs from the other SCMs mentioned in this chapter by not being an industrial by-product. Consequently, its environmental footprint is greater than that of the others. The principle source of carbon emissions associated with metakaolin production is the heating of kaolin to about 750°C. A recent study by Heath et al. (2014) reported a value of 0.423 tonne CO_2eq/tonne, while quoting values from other studies of 0.330 and 0.370 tonne CO_2eq/tonne.

In addition, the replacement of cement with metakaolin does not greatly reduce abiotic depletion. Indeed, Heath et al. (2014) quoted that 1.16 tonnes of kaolin are required to yield 1 tonne of metakaolin compared to 1.6 tonnes of limestone and clay to produce Portland cement. Furthermore, kaolin mining in the United Kingdom produces large quantities of waste and can impact local aquatic environments (Thurlow, 2005, cited in Heath et al., 2014).

As with silica fume, additional environmental benefits may be derived from the use of metakaolin in terms of increased durability and improved strength (albeit greater than for silica fume in the former case and less in the latter). Again, these effects are project-specific, so generalisations cannot be made here.

17.7 Future trends

With global cement clinker production at 3.57 Bt per year, and rising, there is a need for an increased use of SCMs, if concrete is to continue in its drive towards lower CO_2 emissions. However, because many SCMs are industrial by-products, their availability depends on the production of the respective primary products, that is, iron production for GGBFS or burning of pulverised coal for PFA. Annual production of about 1.7 Bt of steel (including 1.2 Bt from pig iron) equates to about 500 Mt of blast-furnace slag. Meanwhile, global production of PFA was approximately 660 Mt in 2010. Combined, this equates to a possible replacement level of just below 33%.

However, global figures hide huge regional variations, both in production and in the use of cement replacement materials. Almost half of the world's steel is produced in China, with Japan, the United States and India being the only other countries to produce more than 5% each of the global total. China, India and the United States also feature in the top five global cement producers, accounting for 62%, 7% and 2%, respectively. Globally, almost all blast-furnace slag is reused, and there are shortages in some countries.

As with cement and GGBFS production, China is the world's leading consumer of coal, producing about 340 Mt of PFA per year, just over half of the global figure. The United States and India are the next largest consumers, producing 140 and 90 Mt of PFA each per year, 21% and 14%, respectively. However, the figures for PFA use in concrete are much more variable than those for GGBFS. Over 90% is reused in Japan and the European Union, 67% in China and less than 20% in the Middle East, Africa, Russia and India (Heidrich et al., 2013). There remains, therefore, considerable potential for the greater global use of PFA in concrete. In the United Kingdom, overall replacement levels are about 13% (Mineral Products Association, 2013), but rising. The United Kingdom's production of cement is about 10 Mt per year, with production of GGBFS about 2 Mt and PFA about 2 Mt, which leaves considerable room for increased replacement levels.

However, external factors may impact the availability of SCMs for inclusion in concrete.[2] Global cement production has increased by about 10% per year on average since 1990. Meanwhile, growth in global steel production has been approximately 5% per year over the same time period. Much of this growth in both materials has occurred in a limited number of countries. For example, in 1990 China produced 63 Mt of steel and 210 Mt of cement, equivalent to 11% and 20% of the respective markets. In 2014 this had risen to 711 Mt of steel and 2480 Mt of cement, 57% and 69% of the respective global markets. Similar increases, albeit on a lesser scale, have occurred in India.

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[2] This section has concentrated on threats to the production of GGBFS and PFA, given that these are the current principal SCMs used worldwide.

Therefore, it may be considered fortuitous that regional growth in cement production has been coupled with growth in the availability of GGBFS (although the relationship between the two is due to more than fate alone). Meanwhile, as the iron industry diminishes in a number of countries, the availability of GGBFS becomes problematic. For instance, the US Geological Survey states that GGBFS is increasingly difficult to source in the United States (USGS, 2015c).

Furthermore, there are technical developments which may also impact the availability of GGBFS. Direct reduction of iron (DRI), a technique used for many hundreds of years, has become increasingly available on a large industrial scale. In the process, iron ore is reduced to iron by heating in a reducing atmosphere (typically, a mixture of hydrogen gas and carbon monoxide) at 800–1050°C. The resultant product is often known as sponge iron, because a high surface area is required in order to achieve rapid ore reduction, resulting in a porous product. India is currently the world's biggest producer of sponge iron, producing 20 Mt in 2014, which is 27% of India's steel production. The use of DRI has increased steadily over the past 30 years, and now accounts for 6% of global iron, or 13% of non-Chinese iron.

Information on the composition of the by-products arising from DRI is difficult to find, although the sponge iron is often discharged directly into an EAF such that EAF slag may be considered as the relevant by-product. This has implications when considering its use as an SCM. Firstly, while production of 1 tonne of iron in a blast furnace may yield 300 kg of GGBFS, only about 150 kg of EAF slag is produced. Secondly, the composition is a lot more dependent on the precise conditions within the furnace, potentially leading to greater variability. EAF slags often have similar calcium contents to GGBFS (35–45%) but have much lower silica contents and higher iron contents. This makes EAF slags typically unsuitable for use as an SCM because of a potentially high free-lime content and low pozzolanicity. The growth of DRI may reduce the availability of GGBFS for use as SCMs.

As with growth in steel and cement production, the use of coal has experienced an inexorable growth in recent years. Global coal production increased from 4.7 Bt in 1990 to 7.8 Bt in 2012, with much of it used for generating electricity. With current consumption yielding about 660 Mt of PFA per year and global production increasing, threats to the long-term availability of PFA seem limited. However, concerns are increasing about the carbon intensity of the generation of electricity by coal combustion. This has led to the implementation of regional policies, for example, replacement of coal with biomass in some UK power plants. This had implications for the United Kingdom's construction industry in the summer of 2014 (New Civil Engineer, 2014) when reduced electricity demand coupled with increased construction activity resulted in a shortage of PFA across the United Kingdom. Although this was just an isolated incident, and there are considerable stockpiles of PFA which could be valorised if necessary, a widespread shift away from coal for electricity generation could lead to such events becoming more common. Note that this is not an argument in favour of coal combustion for electricity generation, which releases more carbon dioxide than the savings offered by the use of PFA as an SCM.

Another threat to the continued use of SCMs in construction arises from an unlikely source: life-cycle analysis methodology. As previously mentioned, during a life-cycle assessment, allocations currently follow the primary product. Although this methodology is unlikely to change, some people are beginning to question whether, since civil engineers specify GGBFS because of the improvements in technical performance it offers, GGBFS should continue to be considered as a by-product. The question would then arise as how to partition emissions between the iron and the GGBFS to reflect the slag's position as a product rather than a by-product.

Although there may be many ways to allocate emissions among different products from the same process, a common convention is to partition based on a mass-balance approach. The embodied carbon of 1 tonne of iron is 2.03 tonne CO_2eq/tonne (Hammond and Jones, 2008), and 1 tonne of iron results in the formation of about 270 kg of GGBFS. Partitioning the embodied carbon such that 21.3% of the total emissions $(0.27/(0.27+1))$ are allocated to the GGBFS gives a value of 0.43 tonne CO_2eq/tonne, to which the 0.067 tonne CO_2eq/tonne associated with granulating and grinding the GGBFS should be added, giving a total of about 0.5 tonne CO_2eq/tonne (while reducing the embodied carbon of iron from 2.03 to 1.6 tonne CO_2eq/tonne). Although the revised value for GGBFS is lower than that for Portland cement, a seven-fold increase in embodied carbon is likely to make use of GGBFS as an SCM less attractive. If this were to consequently reduce the use of GGBFS as an SCM, the carbon savings would be entirely hypothetical as civil engineers switch from using slag cements to higher carbon alternatives. A similar calculation performed for PFA would be more difficult, yet would undoubtedly increase the embodied carbon of PFA and discourage its use as an SCM.

More recently, there has been another development. As an alternative to increasing the use of SCMs, some researchers have started to look at ternary blends, reducing the clinker content of the cement by blending with more than one SCM or blending an SCM with a reactive mineral addition such as limestone. Still in its developmental stages, this approach is beyond the scope of this chapter, but clinker contents of 50% appear possible.

In summary, the use of SCMs in concrete is an efficient means of reducing the environmental burden of construction. A range of materials are available, including industrial by-products (GGBFS, PFA and silica fume), natural pozzolans and calcined materials (metakaolin). The industrial by-products offer three environmental benefits: reducing embodied carbon, reducing abiotic depletion and diverting an otherwise waste material from landfills. The natural pozzolans and calcined materials still offer a reduction in embodied carbon, but without the other benefits. Furthermore, many of the SCMs offer improved durability, therefore improving environmental performance over the lifetime of a structure.

Continued growth in the production of cement is matched by growth in the global production of steel and the consumption of coal. As such, there are ample supplies of the principle SCMs to maintain average replacement levels above 30%. However, the adoption of methods for lowering the use of energy in steel production or the transition to lower carbon electricity generation, while laudable from a societal perspective, may place pressure on the availability of SCMs in the future. Consequently, a transition from GGBFS and PFA to calcined clays or to the development of ternary blends to maintain performance at low clinker contents without excessive SCM consumption may be more popular in the future.

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Sustainability of alkali-activated cementitious materials and geopolymers

18

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18.1 Introduction

Alkali-activated cementitious materials (AACMs) and geopolymers represent a family of alternative binders for the full or partial replacement of Portland cements. An AACM is essentially a material that gains strength by means of a chemical reaction between a source of alkali, commonly an alkali silicate solution, and an aluminate-rich material. As well as being highly sustainable, largely composed of industrial by-products already familiar to the concrete producer, such as fly ash and blast furnace slag, they can exhibit performance characteristics beyond those of conventional Portland cement-based materials. Enhanced chemical, wear, and temperature resistance have all been shown to be achievable. Bearing more resemblance to a natural stone than a cement, these materials promise low-carbon, high-performance alternatives to current binder technology.

The term 'geopolymer', first used in 1979 although the process is clearly much older, describes binders produced by the polycondensation of alumina-silicates such as fly ash and slag with alkali activators to produce inorganic polymers (Davidovits, 1979). The large-scale production and use of geopolymer-type materials can be traced to the immediate post–World War II Soviet Union where 'soil cements' were employed in the construction of high-rise apartment buildings, which have subsequently proved to be very durable.

While there are distinctions between AACMs and geopolymers, for end-users the terms are often interchangeable. Wider take-up of such materials has to date been restricted by the lack of track record and a dearth of nationally and internationally recognised standards to define their characteristics and performance. Practical applications with associated evidence of performance, plus standards and guidance on their use, are starting to become available (RILEM TC 224, 2014; BSI, 2016). These should allow such materials to be more widely adopted in larger scale commercial applications.

Many formulations remain alkaline in nature and capable of working well with conventional reinforcement, although it may be some time before sufficient data are available to predict the long-term corrosion behaviour of embedded steel in more aggressive environments, such as marine applications or where exposed to deicing salts.

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The sustainability credentials of such materials may be expected to be of the greatest interest to potential adopters of the technology. For the main component in the binder, the energy commitment has already been made. Relatively little additional energy is required to convert them to useful materials and significant savings of energy and carbon equivalent, up to 80%, are often claimed over Portland cement equivalents (RILEM TC 244). However, as most modern conventional concrete contains a large proportion of the same components, such as blast furnace slag (BFS) and fly ash, as cement replacements, simple energy input comparisons can sometimes be misleading.

The chemical resistance of certain of these materials can be superior to their conventional counterparts so their use in wastewater and other potentially aggressive applications should prove attractive. Unlike traditional Portland cement-based materials, the structure of the hardened geopolymer does not rely on hydrates, making it much more stable at high temperatures and potentially imparting other advantages over their conventional counterparts.

Unlike some earlier versions that required heating to cure, many AACM formulations are ambient curing, allowing their use in most if not all forms of construction. They can be placed, poured, or sprayed in a similar manner to any conventional material and have been successfully trialled for precast applications (see Section 18.5).

The technology of AACMs and geopolymers promises low-energy/low-carbon materials with enhanced temperature and chemical resistance properties. They provide most of the attributes required from conventional Portland cement-based materials, potentially delivering the levels of durability and sustainability that modern construction practices increasingly demand.

18.2 The manufacture of inorganic polymers

18.2.1 Raw materials

The raw materials of AACMs and geopolymers fall into three categories. The first category produces low-calcium, alkali-activated systems, which include materials often described as geopolymers. A common precursor in this category is fly ash, which has relatively high alumina (Al_2O_3) and silica (SiO_2) content. The second category is based on high-calcium precursors such as BFSs that have higher calcium oxide (CaO) content than fly ash but less than Portland cement. The third category is hybrid binders that use mixtures of precursors.

18.2.1.1 Low-calcium precursors

Low-calcium, alkali-activated binders constitute raw materials rich in aluminosilicates, alumina (Al_2O_3), and silica (SiO_2). A prime example is lower calcium fly ash of Class F, which are highly pozzolanic. The chemical composition of a typical Class F material constitutes over 70% of the pozzolanic compounds SiO_2, Al_2O_3, and Fe_2O_3. However, it is not only the fly ashes derived from traditional black coal that are suitable for alkali activation to produce geopolymers. Low-calcium ashes produced from the combustion of brown coal and fluidised bed ash from coal combustion are also suitable for this purpose. Rice husk combustion ashes, which are rich in SiO_2, are also potential raw materials (RILEM TC 224, 2014).

Although low-calcium fly ashes are generally considered to produce superior geopolymer performance, the role of Ca content is not straightforward (Oh et al., 2010). The calcium content of Class C fly ashes, usually produced from lignite and subbituminous coals, is greater than Class F fly ashes and exceeds 20%. In some studies, high-calcium ashes have shown greater strength development while others have shown the opposite (RILEM TC 224, 2014).

18.2.1.2 High-calcium precursors

Ground granulated BFS from the steel industry is a commonly used precursor for alkali-activated materials (Shi et al., 2006). The high calcium level is provided by the CaO component that typically ranges between 31% and 46% by weight. Other major compounds and their typical contents are SiO_2 (30–40%), Al_2O_3 (6–18%) and MgO (2–18%) (EUROSLAG, 2003).

The optimum fineness of BFS for use in alkali-activated materials ranges between 400 and 550 m^2/kg (RILEM TC 224, 2014). The glass content of slag used in blended Portland cement is 90–100%. Glassy BFS with CaO/SiO_2 ratios between 0.5 and 2.0 and Al_2O_3/SiO_2 ratios of 0.1–0.6 are suitable precursors for alkali activation (Talling and Brandstetr, 1989).

18.2.1.3 Hybrid binders

The development of hybrid binders to combine the individual benefits of low-calcium and high-calcium alkali-activated materials has gained momentum in recent years. High strength and durability together with low cost and sustainability benefits are providing the incentive for developing these binders. The blending of fly ash with BFS produces a suitable hybrid binder where the calcium from the BFS and both alumina and silica from the fly ash aid the alkali-activated reactions of the binder. The nature of the reactions is not currently fully understood. The replacement levels of BFS with fly ash in the blended binder can be optimised for different chemical compositions of each precursor to achieve best performance. Some other constituents that can be used in hybrid binders include metakaolin, silica fume, calcined clays, and calcium aluminate cement (RILEM TC 224, 2014).

The performance of BFS-based, alkali-activated materials under high-temperature exposure is improved by adding metakaolin to the system. Calcium aluminate cement can be blended with calcined clays, natural zeolites, or pozzolans (Fernández-Jiménez et al., 2008) to improve strength and to control efflorescence (RILEM TC 224, 2014). Metakaolin plays an important role in hybrid blends with various fly ashes and slags by supplying Al to the reaction process. The properties of alkali-activated fluidised bed combustion fly ash can be improved by blending with silica fume and $Al(OH)_3$. The high silica content of silica fume reacts with the alkali solutions and CaO to form additional calcium silicate hydrate.

18.2.2 Activators

When mixed with water, BFS undergoes a low rate of hydration and develops strength slowly. Exposure to an alkaline activator creates a high pH environment and accelerates the hydration reactions. Commonly used activators are industrial products whose manufacturing technology is well established and are manufactured in large quantities and therefore easily available for supporting new applications.

Sodium silicate (Na_2SiO_3) and sodium hydroxide (NaOH) are commonly used activators. Sodium silicate solution is available in different grades defined by its silica modulus or SiO_2/Na_2O ratio. Sodium hydroxide (NaOH) is produced as a solid by the electrolysis of sodium chloride solution, which produces NaOH, hydrogen and chlorine. Solid NaOH is dissolved in water to provide a solution of the required molarity.

Sodium carbonate and sodium sulphate have also been used as activators for BFS. They are less reactive than the silicate-based activators and strength development is slow. Other silica-rich sources can also form suitable activators. For example, silica fume combined with NaOH activator when used with BFS binders produces a high-performance, alkali-activated material that is comparable to sodium silicate activated formulations (Živica, 2006). Other high-silica sources that form potential activators are rice husk ash and nanosilica (RILEM TC 224, 2014). The use of an alkali silicate with finely ground limestone as activator for a BFS binder has the potential of making an economical alkali-activated material of moderate performance (Sakulich et al., 2009).

18.2.3 Alkali activation reactions

The two general cases of low-calcium precursors, such as fly ash, and high-calcium precursors, such as BFS, are considered. BFS is more reactive than fly ash at moderate alkaline pH levels and, therefore, can be used with a greater range of activators in addition to the hydroxides and silicates that are suitable for fly ash. Low-calcium precursors, typically fly ashes, are rich in S_iO_2 and Al_2O_3 compounds, whereas high CaO content of high-calcium precursors dominates the hydration reactions.

Alkali activation of a low-calcium precursor produces an amorphous aluminosilicate gel, which resembles the structure of a zeolite. The gel also contains small amounts of minerals such as quartz present in the original fly ash. Aluminium and silicon atoms are arranged in the cross-linked framework of the tetrahedral structure of the gel. The negative charge of the SiO_4 and AlO_4 groups is balanced by alkali ions such as sodium. The water in the low-calcium, alkali-activated fly ash does not take part in the hydration reactions since there is no formation of $Ca(OH)_2$ to activate pozzolanic reactions.

The products of alkali activation of BFS are influenced by the chemical constituents of the slag and the activator, such as Ca, Mg, Si, Al, Na, S, in addition to the pH, liquid/binder ratio, and curing. The dominant hydration phase is the calcium silicate hydrate (C-S-H) broadly similar to the tobermorite gel formed in hydrated Portland cement. The lower Ca content of slag compared to Portland cement and the greater Al substitution in the tetrahedral structure leads to cross-linking between tobermorite chains (Provis, 2014). The complex hydration system of alkali-activated slags can be simplified into the following categories:

> N-A-S-H gel is formed in alkali-activated slags with low Ca+Mg content. A secondary phase of crystalline zeolites is also present.
> C-A-S-H gel is formed in alkali-activated slags with high Ca content. Na ions are also present in the gel at charge-balancing sites and the gel may be defined as C-(N)-A-S-H. AFm (monosulphate) in crystalline form appears as a secondary phase if sufficient Al is available.
> N-A-S-H gel exists as a secondary product with the C-A-S-H gel in most chemical compositions of slags and in hybrid binders involving high and low Ca precursors.

The main difference in the above hydration reactions of alkali-activated binders and ordinary Portland cement is that in the latter case chemical reactions of hydration are activated by water. The main hydration products are the C-S-H tobermorite gel and calcium hydroxide, with AFm (monosulphate) and AFt (ettringite) as secondary phases.

18.3 Fresh properties

One of the challenges for this technology has been to achieve scaled-up production of alkali-activated concrete with sufficient workability and working time. AACMs generally tend to be more cohesive than Portland cement concrete and stiffen rapidly. The numerous workability and plasticising admixtures available for normal concrete are not specifically designed for compatibility with the AACM chemistry. The efficiency of the few that can be used with AACMs is less than in Portland cement-based binders. However, commercial compositions of AACMs suitable for large applications are available, and there are many examples of their use. Working time has been managed and there are examples of compositions developed for structural applications including repair (Science Daily, 2009; Concrete Repair Bulletin, 2015; Lambert et al., 2015). The future scope for developing admixtures tailored for AACMs is promising and, in turn, will open up many more opportunities for the use of alkali-activated materials.

The density of the fresh and hardened AACM concrete is influenced by the specific gravity of binder materials, the aggregates, and their mix proportions. The lower specific gravity of precursors such as fly ash can reduce the density to some extent but it remains in a similar range to Portland cement-based concrete.

18.4 Mechanical properties

18.4.1 *Strength and stiffness*

Compressive strengths greater than 50 MPa have been reported in many laboratory evaluations of alkali-activated, BFS high-calcium binders and fly ash-based low-calcium binders. Higher strengths up to 110 MPa have also been achieved (Talling and Krivenko, 1997). Typically, high-temperature curing for relatively long periods has been required to activate the hydration process and achieve such values. Steam curing has also been used to develop good compressive strength in precast members (Sumajouw et al., 2007; Talling and Krivenko, 1997). However, a well-designed AACM can cure at ambient temperature and develop high compressive strength (Science Daily, 2009). Such materials cure at ambient temperature to give 28-day compressive strengths exceeding 65 MPa. A typical modulus of rupture value for this material is 9.5 MPa, which is over 10% of its compressive strength. The relationship between compressive strength and age of the AACM concrete cured in air (20°C, 60 RH) is plotted in Fig. 18.1 for two compositions with different aggregate grading and content. The compressive strength under wet curing can be somewhat lower but occasionally it is higher. In practice, wet curing in the early period followed by air curing is desirable to prevent cracking, control shrinkage, and optimise strength development.

Fig. 18.2 shows the shear-bond strength of the hybrid AACM mortar (Science Daily, 2009) used to bond a sandstone façade to a medium-strength breeze block. The bond tests were conducted after different durability exposures given in the figure. A comparison of Figs. 18.2 and 18.3 shows the significantly superior shear-bond capacity of the AACM mortar relative to a common proprietary multipurpose epoxy adhesive particularly at very high temperature exposure.

The behaviour of AACM concrete under compression is similar to conventional Portland cement concrete giving comparable values for strain at failure and the relationship between elastic modulus and strength. However, more detailed relationships between compressive strength and such parameters need to be developed for the many variations of potential compositions of AACM structural concrete. Some contradictory results have been reported on the influence of Si/Al ratio on elastic modulus (RILEM TC 224, 2014). The Poisson's ratio of geopolymer concrete under

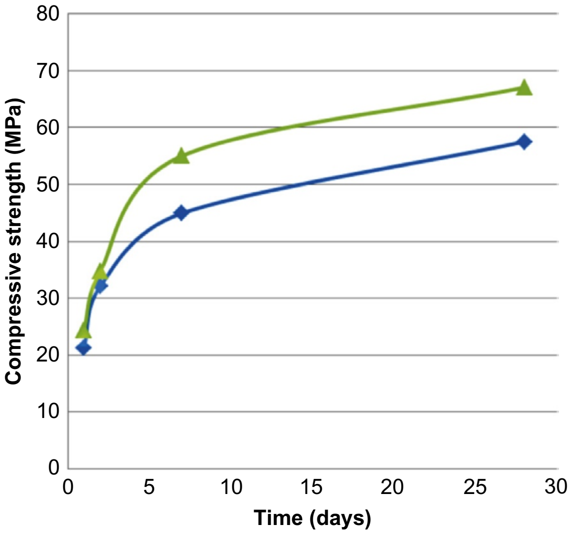

Fig. 18.1 Relationship between compressive strength and age of hybrid AACM concrete (Mangat, Unpublished data).

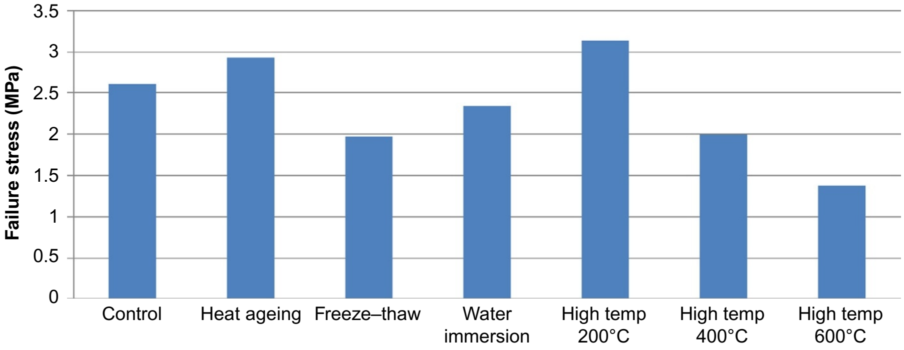

Fig. 18.2 Shear-bond strength of a hybrid AACM applied to a medium-strength breeze block (Mangat, Unpublished data).

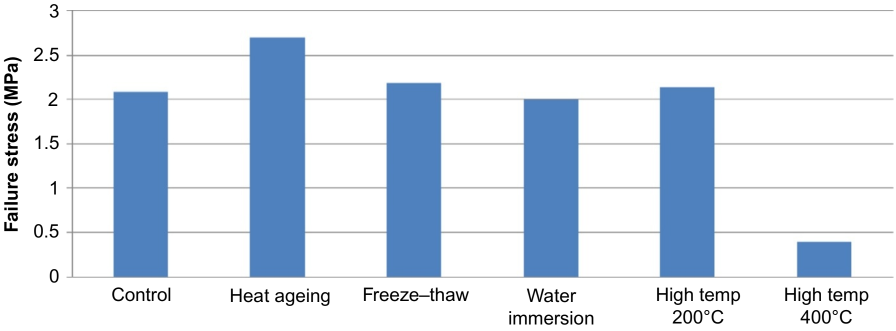

Fig. 18.3 Shear-bond strength of multipurpose epoxy adhesive applied to a medium-strength breeze block (Mangat, Unpublished data).

compression is similar to Portland cement-based concrete with compressive strengths from 40 to 90 MPa. Typical values range between 0.12 and 0.16.

The relationship between modulus of rupture and compressive strength of AACM concretes shows a higher flexural strength relative to Portland cement-based concretes. This is possibly an indication of the stronger bond at the aggregate–matrix interface due to improved pore structure of the interface zone.

18.4.2 Shrinkage

AACMs are typically prone to higher drying shrinkage, relative to Portland cement-based binders, when manufactured and cured under ambient temperatures. The shrinkage reduces by an order of magnitude under high-temperature curing conditions such as are commonly adopted for AACMs. The higher shrinkage is due to the lower degree of chemical binding of water in AACM hydration gels compared with Portland cement hydration products. The shrinkage trends are variable with different precursors and activators. It generally increases with increasing activator content and increasing activator silica modulus.

The factors that help to control shrinkage are good mix design of the AACM concrete, resulting in high aggregate content that restrains shrinkage, adequate early age curing preventing rapid drying, plus the use of shrinkage-reducing admixtures (Mangat and Ojedokun, 2016; Dunster, 2013). Fibre reinforcement of AACMs also offers a mechanism for reducing shrinkage and providing crack control (Mangat & Bordeian, in press). The use of lightweight aggregate and other porous aggregates is also beneficial for shrinkage, while providing a strong chemical interaction with the binder at the interface. Fig. 18.4 shows the shrinkage development with age of two hybrid AACM concrete compositions (Science Daily, 2009), giving a typical range of shrinkage values from tests conducted according to BS ISO 1920-8:2009, Part 8.

AACM compositions are also prone to microcracking at early ages, before adequate tensile strength has been developed. The problem is aggravated with high paste (binder) content and dry curing, as would also be the case with Portland cement-based matrices. High activator content and high modulus of activator increase the risk and extent of microcracking.

18.4.3 Creep

The creep behaviour of AACMs is different from Portland cement-based concrete, and is caused by differences in microstructure, water held in the gel structure, and the rate of strength development. The longer-term creep deformation of AACM concrete, beyond 90 days, continues to increase at a greater rate than Portland cement concrete.

The limited data available in the literature indicate that creep of BFS-based AACM concrete is lower with silicate activation compared with hydroxide activation and is highest with carbonate activation (Talling and Krivenko, 1997). Similarly, limited data available on low-calcium (fly ash)-based alkali-activated concrete show little influence of curing temperature (oven or steam curing) on creep. The specific creep

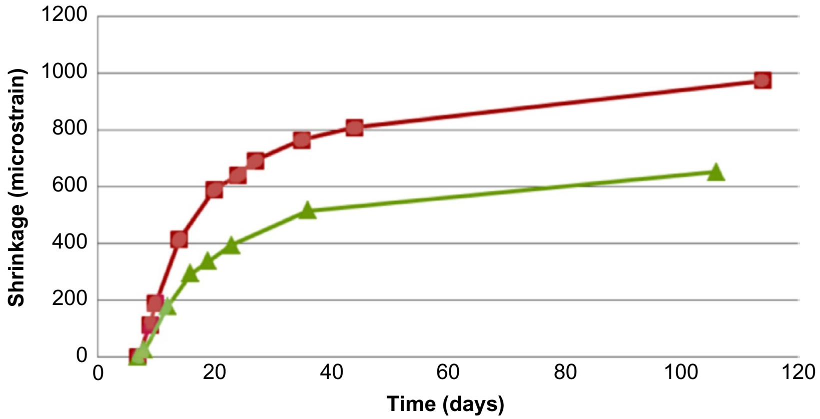

Fig. 18.4 Relationship between drying shrinkage and age of hybrid AACM concrete (Mangat, Unpublished data).

of AACM concrete is reported to be lower than Portland cement concrete of similar grade and falls within acceptable limits defined by the design codes (Rangan, 2014; Sagoe-Crentsil et al., 2013).

18.5 Physical and durability properties

18.5.1 Permeability and porosity

The distribution of capillary pores in the AACM matrix is important since it controls the mass transport mechanisms of deleterious substances such as chlorides and sulphates. While mercury intrusion porosity can provide useful insights to this property, relatively limited information on the pore structure of AACMs is available in the literature. The current state of knowledge indicates that the pore size distribution of AACMs falls into two separate zones of pore size >1 μm and <20 nm range, with insignificant porosity in between. In contrast, similar grades of Portland cement-based matrices have a unimodal pore size distribution of 10–100 nm. The impact of this difference in the pore structure of AACMs relative to normal concrete may be significant with respect to the durability properties of AACMs and require more research. The type of activator also has an effect, with potassium-based activators reducing the median pore diameter more effectively than sodium-based activators.

As with good practice in concrete technology, porosity is reduced by high-quality curing, such as sealed conditions and longer periods of hydration. Generally, the sorptivity of AACMs is within a comparable range with similar grade concretes (RILEM TC 224, 2014). The capillary sorptivity is reduced by employing a lower water content and a higher silica modulus activator.

One potential advantage of AACMs relative to normal concrete is likely to be in the aggregate–matrix interfacial zone. This zone in Portland cement matrices tends to be more porous and weaker than the bulk matrix remote from the aggregate surface. The alkali activator in AACMs, however, may be expected to interact chemically with the aggregate particles in addition to the binder. There is already evidence available of this effect but further research is required for validation.

18.5.2 Chloride, carbonation, and corrosion

The mechanisms of chloride diffusion, carbonation, and steel reinforcement corrosion in normal concrete form the starting point for understanding such mechanisms in AACMs. The basic difference is the pore structure and the chemistry of the pore fluid in AACMs relative to normal concrete. Chloride diffusion test methods that measure the progress of chloride penetration directly, such as the BS EN 12390-11 (BSI, 2015) and the older Nordtest (Nordtest, 1995), give a more reliable comparison of chloride diffusion between AACMs and concrete. Such test methods that analyse powder samples to determine soluble chlorides (water or acid) are relatively independent of the pore fluid chemistry of the AACM binder. The pore fluid chemistry of AACMs leads to inconclusive results on chloride diffusion and requires more research. Chloride diffusion tests based on these standards have been conducted on the hybrid AACM formulations, which were cured under ambient conditions (Science Daily, 2009). The results show superior chloride resistance of the AACM concrete compared with a similar grade of normal Portland cement-based concrete (Mangat and Ojedokun, 2016).

Pore fluid provides the electrolyte for both the protection and the corrosion processes of steel in AACMs and it is important to understand its chemical composition. AACMs can be based on a wide range of precursors and activators each of which will vary the pore fluid chemistry. The presence of threshold levels of chloride concentration at the steel interface initiates corrosion in normal concrete. The chemistry and, in particular, the buffered high pH of the pore fluid provides passivation to steel corrosion until chlorides exceeding the threshold level cause the initiation of corrosion. The $Ca(OH)_2$ buffer is limited in AACM concrete and its extent depends on the hydration products of the particular precursor used. The presence of sulphide in AACM binders using BFS influences steel corrosion differently to the corrosion mechanism in normal concrete (RILEM TC 224, 2014). Service life prediction models and accelerated durability test methods used for concrete will not be directly valid for AACM concrete due to these differences.

AACMs based on BFS and blends with metakaolin show higher degrees of carbonation relative to conventional cement and mortar due to the lack of $Ca(OH)_2$ in the hydration products. However, a suitable choice of activator and its concentration can mitigate this effect. Mix design of AACM concrete is an important factor in controlling carbonation. Carbonation depths comparable to similar grade concrete can be achieved by providing higher binder (paste) contents in AACM concrete mixes.

Carbonation results of AACMs under accelerated carbonation testing often show inferior performance to Portland cement binders, which contradict data obtained under long-term natural exposure. There are some field data on carbonation of AACMs available for structures built in Russia, Ukraine, and Poland under a service life of over 40 years. Carbonation rates of under 0.5 mm/year are indicated for these climates. Carbonation rates of under 1 mm/year were obtained with the standard phenolphthalein test that is considered unsuitable for AACMs (RILEM TC 224, 2014).

The phenolphthalein test for measuring carbonation depth was developed for normal concrete and it is compatible with the chemistry of ordinary Portland cement hydration products ($CaOH_2$). The application and interpretation of this test to AACMs and to modern blended cements requires investigation, and alternative test methods will need to be developed.

Carbonation induced reinforcement corrosion in concrete is considered to occur when the carbonation front reaches the steel surface, reducing the high alkalinity of the pore fluid through the conversion of calcium hydroxide ($CaOH_2$) to calcium carbonate ($CaCO_3$). The reduction in pH depassivates the steel and can allow the initiation of corrosion. This process is unlikely to be replicated in AACMs due to the different hydration products, notably to absence of $CaOH_2$, and other differences in the chemical composition of the pore fluid. In addition to reinforcement corrosion, the other deleterious effect of carbonation in Portland cement concrete is carbonation shrinkage. The carbonation shrinkage property of AACMs is currently unknown and requires further investigation.

The effect of accelerated exposure to carbonation is more severe on AACMs than on normal concrete. The correlation of accelerated test data with natural carbonation under service conditions is also not clear. This is related to the difference in the CO_2 diffusion process and the chemical composition of the pore fluid relative to Portland cement binders where the $Ca(OH)_2$ concentration is high. The rate of carbonation in AACMs is strongly affected by the CO_2 concentration of the exposure environment. The rate of pore fluid carbonation and degradation of the gel pore structure change significantly at high CO_2 exposure levels and the ratio of carbonate to bicarbonate is affected (Puertas and Palacios, 2007; RILEM TC 224, 2014). Accelerated carbonation testing at CO_2 concentrations above 1% is not recommended.

18.5.3 Efflorescence

Efflorescence in AACMs is due to three factors: porous microstructure of some AACM compositions, high alkali concentration in the pore fluid such as in sodium aluminosilicate binders activated with high Na_2O/Al_2O_3 solutions, and relatively weak binding of Na in the aluminosilicate gel structure. Replacement of sodium with potassium improves binding with aluminosilicate gel, and increasing the reactive aluminium content provides more alkali-binding capacity. However, low permeability is an important requirement for a durable material and an appropriately formulated and cured AACM will be more resistant to efflorescence (RILEM TC 224, 2014).

18.5.4 Freeze–thaw resistance

The freeze–thaw resistance of Portland cement concrete is related to physical characteristics such as pore structure, pore saturation, strength of the matrix, and voids provided by air entrainment. The chemical composition of the hydration products is less significant. The same factors are likely to control the freeze–thaw resistance of AACMs. The freezing temperature of the pore fluid, however, will be affected by the differences in ionic composition of the pore fluid and by the relative degree of its confinement within the pore structure of Portland cement and AACM binders. The freezing point of pore fluid in a BFS-based AACM has been reported as significantly lowered to below −50°C due to its high ionic concentration, resulting in higher freeze–thaw resistance than a Portland cement-based binder. Many studies showing satisfactory frost resistance and resistance to frost-salt attack of AACMs have been reported. Data from in-service structures have shown better frost resistance of AACM concrete compared with similar grade conventional concretes exposed to the same conditions (Davidovits, 2008; Rostovskaya et al., 2007). Early age freezing and thawing is less damaging to AACM concrete than to Portland cement concrete, and freezing is detrimental only before the AACM concrete has reached 5 MPa strength. The freeze–thaw resistance of fly ash-based AACMs also meets the specifications for cold weather applications.

Some studies have shown worse freeze–thaw resistance of AACM concrete than similar grade Portland cement concrete (Bilek and Szklorzova, 2009). This is attributed to the higher amount of free water present in the pores of AACMs due to the lack of crystal phases, such as $CaOH_2$ and ettringite, which are present in Portland cement hydration products. These crystal phases contain the water in chemically and physically bound form and prevent it from freezing. This effect is aggravated in AACMs by the presaturation of test specimens as required by testing standards before exposing the specimens to freeze–thaw conditions. Such binders can thus contain more water in the pore network within the gel than Portland cement binders, which then undergo freeze–thaw cycles. This can result in more freeze–thaw damage if the tests are conducted near the water saturation level of the specimen. The freeze–thaw test methods given in most standards are expected to be suitable for AACMs but any precuring phase that requires presaturation of the test specimens needs to be reviewed.

The freeze–thaw and scaling resistance of AACMs can be reduced by carbonation and microcracking due to restrained shrinkage. Carbonation of the matrix makes it more brittle and carbonation shrinkage can lead to cracking and scaling.

18.5.5 Sulphate and acid resistance

Low-calcium binder-based alkali-activated material cured at 60°C and exposed to sodium sulphate solution showed excellent sulphate resistance. Portland cement binders can suffer sulphate attack due to chemical reactions with the hydration products resulting in the formation of gypsum and subsequently ettringite that is expansive and can result in disintegration of the concrete. The hydration products of alkali-activated, low-calcium precursors such as fly ash are relatively free from C-S-H, and sufficient expansive ettringite is not formed to cause damage. High-calcium precursors such as BFS can produce more expansive ettringite.

The acid resistance of alkali-activated, low-calcium binders is similarly superior to Portland cement concrete due to the difference in hydration products and ettringite formation.

18.5.6 Fire resistance

The high-temperature resistance of AACMs based on low-calcium binders (geopolymers), with fly ash as the precursor, has been widely reported in the literature (RILEM TC 224, 2014). Their amorphous gel structure is considered to provide fire resistance by preventing the buildup of internal pressure, which leads to violent splitting and spalling of Portland cement concrete when exposed to fire. A hybrid AACM composition has passed the Eurocode (EN 1991-1-2:2002) fire test at temperatures over 1100°C (Science Daily, 2009). In addition, a fireplug test demonstrated satisfactory insulation to services while satisfying the design load capacity of 5 kN/m^2 (Mangat, 2009). Fig. 18.5 shows a furnace test sample of the same AACM composition, displaying its typical response to high temperature. Comparative fire tests against similar strength Portland cement concretes demonstrate the release of steam from the gel structure of the AACM concrete, preventing spalling, while the Portland cement concrete spalls explosively.

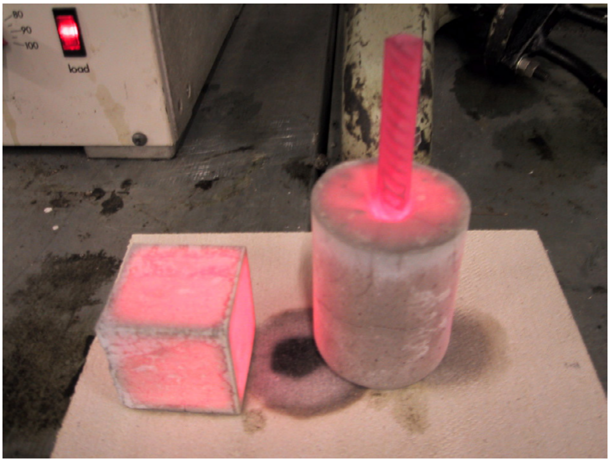

Fig. 18.5 Heat resistance of AACM (Mangat, Unpublished data).

Comparative tests on fly ash and metakaolin-based AACMs exposed to high temperature have shown that the strength of the metakaolin-based AACM decreased after heating, whereas the fly ash-based AACM had higher strength (Kong and Sanjayan, 2008; Kong et al., 2007). This was attributed to the differences in pore distribution in the two materials that allowed the steam to escape more easily and without damaging the microstructure in the fly ash-based geopolymer. Fire tests up to 800°C on metakaolin-based AACMs and blends with BFS showed a higher residual strength (after firing) for the blended materials (RILEM TC 224, 2014). The residual strength of AACM mortar and concrete increases due to further geopolymer reaction while the matrix-aggregate thermal incompatibility causes strength reduction. The balance between the two controls the final strength after fire testing.

18.6 Structural applications

There are many examples of applications of AACM concretes since the 1950s in Europe, the former Soviet Union, and more recently in Asia, North America, and Australia (RILEM TC 224, 2014). These applications have demonstrated the performance of these materials in rigorous environmental and service conditions. In general, their durability after decades of exposure to conditions such as carbonation, freeze–thaw attack, acid attack, and corrosion of the reinforcement has been judged to be satisfactory and the strength of the matrix has increased with time. Some limitations, such as rapid loss of workability, have also been experienced, which are being addressed in current research and development work.

Development of AACMs has progressed to the stage where well-defined formulation of the binder composition together with good mix design to optimise aggregate content, and appropriate liquid/binder ratios, can produce mixes suitable for structural application. Standard structural elements such as beams and columns have been

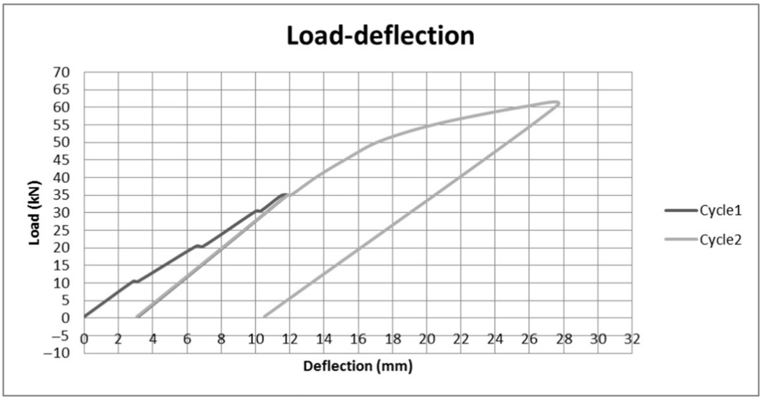

Fig. 18.6 Load-deflection curve of the reinforced AACM lintel (Mangat, Unpublished data).

manufactured and cured at either elevated temperature (steam curing) or at ambient temperature. Test results on these elements show that the strength and deformation characteristics of AACM concrete structural elements are similar to Portland cement concrete elements of the same nominal grade. Reinforced concrete structural design codes can therefore be considered relevant to AACM concrete structures.

A reinforced concrete lintel was manufactured with an AACM concrete composition with similar properties to a structural grade concrete (Science Daily, 2009). The beam was manufactured at a precast plant where the quality control was relatively poor and the reinforcement cage suffered some displacement during casting. A second similarly reinforced concrete beam with Portland cement concrete was manufactured as a control element under better quality control. The beam dimensions were 140 × 213 mm cross-sectional width and depth and a length of 2.7 m. Both beams were manufactured and cured at ambient temperature. Both were tested under four-point bending (span 2.12 m) using the procedure of BS8110. The load-deflection curve of the reinforced AACM concrete beam is shown in Fig. 18.6 and their relative performance is further analysed in the next paragraphs.

18.6.1 Serviceability limit state

18.6.1.1 AACM concrete beam

Instantaneous deflection at serviceability load of (34.5 + 0.455) kN = 11.51 mm

Residual instantaneous deflection upon unloading = 3.15 mm

$$\% \text{ recovery after first loading} = \frac{11.51 - 3.15}{11.51} \times 100 = 72.6\%$$

18.6.1.2 Portland cement concrete beam

Instantaneous deflection at serviceability load of (34.5 + 0.455) kN = 9.62 mm

Residual instantaneous deflection upon unloading = 1.83 mm

$$\%\,\text{recovery after first loading} = \frac{9.62 - 1.83}{9.62} \times 100 = 81\%$$

Hairline flexural cracks were observed in the both beams at serviceability load.

18.6.2 Ultimate limit state

Both beams failed at a load of 60 kN with an ultimate moment of resistance of 30.3 kNm. In comparison, the design ultimate moment of resistance of the beam was 26.1 kNm. The deflection at ultimate load of the AACM and Portland cement concrete beams was 26.0 and 22.3 mm respectively.

The maximum deflection permitted by BS 8110-2: 1985 at the ultimate design lo ad = span/40 = 1/40 × 2120 mm = 53 mm These results are summarised in Table 18.1.

Similar conclusions have been reported for heat-cured (steam-cured) beams and columns of low-calcium-based geopolymer binders (Rangan, 2014). Beam sections of 200 × 300 mm and 3.3 m length were manufactured with tensile reinforcement ratios between 0.64% and 2.69%. The flexural, shear, and bond characteristics of the beams were subsequently investigated. Columns of effective length 1.7 m and 175 mm square cross section were made with 1.47% and 2.95% longitudinal reinforcement. The compressive strength of the steam-cured geopolymer concrete was 40–60 MPa. The structural performance and failure mode of the reinforced geopolymer concrete beams and columns was similar to Portland cement concrete elements of similar grade concrete.

18.7 Future trends

Based on the evidence that structural performance can be equivalent or superior to Portland cement concretes, and a growing understanding of the benefits and limitations with respect to durability and the emergence of standards for their specification, one may expect AACMs and geopolymers to play an increasing role in providing durable and sustainable alternatives to the current products based on traditional Portland-based binders. The use of secondary wastes such as flue gas desulphurisation (FGD) products within the precursors are also potential future developments (Mangat et al., 2006; Khatib et al., in press).

There is already significant interest in their use in challenging environments where resistance to high temperatures or aggressive chemicals is required. The potential savings in, for example, a sewage treatment environment could be considerable if the alternative is to employ conventional concrete with an expensive, chemical-resistant protective coating. Similarly, a resistance to high-temperature exposure resulting in little or no loss of properties, is particularly valuable when selecting materials for tunnel linings.

A major limitation to the more widespread use of AACMs and geopolymers is their lack of track record, especially with respect to the durability of embedded steel reinforcement. While this may restrict the direct replacement of Portland cement binders in conventional reinforced concrete designs, it should also encourage the use of alternative methods of reinforcement. Where the maintenance of a high-alkalinity

Table 18.1 Flexural test results of reinforced beams of AACM and Portland cement concrete, according to BS8110 (Mangat, Unpublished data)

Beam	Instant deflection (mm)	Residual deflection (mm)	Recovery (%)	Moment of resistance (kNm)	Design moment of resistance (kNm)	Ultimate deflection (mm)	Permissible deflection (mm)
AACM beam	11.51	3.15	72.6	30.3	26.1	26.0	53
Control concrete beam	9.62	1.83	81	30.3	26.1	22.3	53

environment is not a necessity for durable service, nonmetallic fibres should offer viable and more sustainable alternatives.

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Sustainable use of vegetable fibres and particles in civil construction

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19.1 Introduction

Sustainability is a concept linked to increasing concerns all over the world in view of the shortage of natural resources and energy, generation of several types of solid wastes, and gas emissions from different sources. The classic definition of sustainable development is that ‘development which meets the needs of the present without compromising the ability of future generations to meet their own needs’ (Drexhage and Murphy, 2010). Acceptance of the report by the United Nations General Assembly gave the term political salience. In 1992, leaders set out the principles of sustainable development at the United Nations Conference on Environment and Development in Rio de Janeiro, Brazil. It is generally accepted that sustainable development calls for a convergence between three pillars: economic development, social equity, and environmental protection (United Nations, 2007). Rational use of vegetable fibre can be an alternative solution to the production of durable and more sustainable goods. Fibres obtained from the various parts of the plants are known as vegetable, or natural, fibres. These fibres are classified into three categories depending on the part of the plant from which they are extracted: blast or stem fibres (jute, malva, flax, hemp); leaf fibres (sisal, pineapple, banana); fruit fibres (cotton, coconut, oil palm); and other (bamboo, wood pulp) (Wood, 1997; Vilaplana et al., 2010; Shah, 2013; Ardanuy et al., 2015; Santos et al., 2015b).

Fibre resources such as bamboo, sisal, green coconut, and those of agricultural residue origin are called natural nonwood fibre resources. These fibres have been used to produce clothing; ropes and cordage; bags; canvas; fishing nets; brushes; sewing thread; as well as pulp, paper, and building products (plaster products, fibre-cement sheeting, particleboards, fibre-reinforced concretes, fibre-reinforced plastics, and insulating materials).

The traditional nonwood fibres can be used to produce high-quality writing and specialty papers (Saijonkari-Pakhala, 2001; González-García et al. 2010; Kamoga et al., 2013). Global paper use increased more than six times over the latter half of the 20th century, and has doubled since the mid-1970s. About 93% of today's paper comes from trees, and paper production is responsible for about a fifth of the total wood harvest worldwide. A sheet of writing paper might contain fibres from hundreds

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of different trees that have collectively travelled thousands of kilometres from forest to consumer. Fig. 19.1 depicts the global production of pulp for paper from wood fibres until 2013 that were most commonly traded internationally.

In recent years, vegetable (nonwood) fibres have been increasingly used as reinforcement in polymer composites. With their low cost and high specific mechanical properties, they can represent a good, renewable, and biodegradable alternative to the most common synthetic reinforcement, that is, glass fibres (Li et al., 2000; Herrera-Franco and Valadez-González, 2005; Doan et al., 2007; Tomczak et al., 2007a; Zini et al., 2007; Vilaplana et al., 2010).

Fibre reinforcement in building materials has been in use since the early ages and its application in civil construction has taken place significantly, with the introduction of asbestos-cement in the world market in the beginning of the 20th century. However, there was not a clear understanding of its durability, mechanical behaviour, and toxicity. Materials based on vegetable plants and alternative cements are well

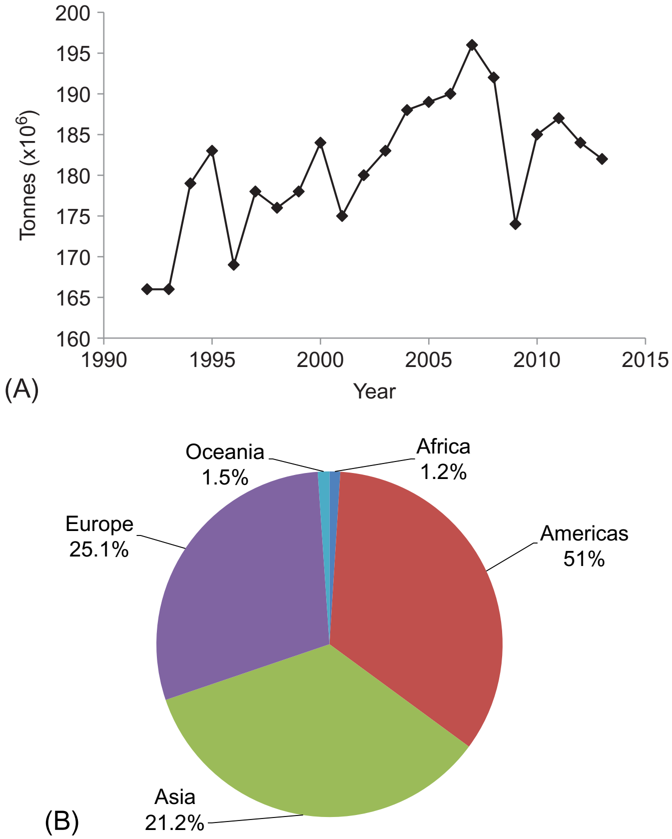


Fig. 19.1 (A) Global production of pulp for paper until 2013. (B) Production by region (FAO, 2015).

known and should be more intensely used for regional solutions or buildings in rural areas. Advantages can involve low cost, energy efficiency, residue and contamination control, thermal comfort, and, principally, the achievement of ecofriendly materials (Coutts, 1988; Agopyan, 1988; Savastano and Warden, 2005; Coutts, 2005; Agopyan et al., 2005; Pereira et al., 2013; Correia et al. 2014).

Particleboard composed of particles of cellulosic material (normally from wood fibres) is a material of great applicability in civil construction, with broad potential for social and environmental sustainability. It stands out among the usual construction materials as the only renewable and biodegradable one, with advantageous properties to form various structural systems, such as walls, floors, and roofs. However, the reduction of forest reserves and the imminent desertification in many tropical countries have directed researchers to find other lignocellulosic fibres or particles as an alternative raw material for production of these particleboards (Fiorelli et al., 2012; Cravo et al., 2015). Agricultural residues, which are annually produced in large quantities throughout the world, are the main renewable resources for particleboard (Zhang and Hu, 2014).

There is also substantial knowledge regarding materials and techniques, which involves earth building containing vegetable plants (nonwood and wood fibres). Unfortunately, the performance codes are mostly focused on modern materials and, in several cases, are difficult to conform to even for special uses (Coutts, 1988; Agopyan, 1988; Plessis, 2001).

The main objective of the present chapter is to discuss the performance of some vegetable fibres in construction applications. The availability of the fibres and their extraction are connected to the manufacturing and processing of these fibrous raw materials for different uses. The chapter also contains an example of application of nonwood fibres as reinforcement in cement-based composites and in matrix of particleboard for building constructions and infrastructure.

19.2 Availability, extraction, and characteristics

There are three basic procedures for extracting nonwood fibres: retting, chemical treatment, and mechanical decortication. The fibre bundles have been traditionally extracted by a microbiological process known as retting in which the combined action of microbial enzymes and water decomposes the nonfibrous material surrounding the fibre bundles and enables the fibre bundles to be loosened for manual extraction (Wood, 1997). The fibres produced by the retting process are still encrusted with high amounts of lignin and hemicellulose, which affect the quality of the fibre. For the production of textiles, the fibres are often subjected to chemical treatment to remove these adhering compounds. Decortication is a process where the fleshy leaves are first trimmed to remove the spines and then passed through decortication machines, which crush them between rollers and scrape them against a bladed drum. During the scraping stages, water is sprayed onto leaves to assist in the separation of fleshy material from the fibre. For some purposes, the blast fibres can be extracted from green or dried stem material simply by mechanical means without the necessity for any pretreatment (Wood, 1997).

Therefore, some advantages and disadvantages of vegetable fibres for building components should be highlighted.

Advantages of vegetable fibres are listed here.

- It has low specific weight, which results in higher specific strength and stiffness than glass. This is a benefit especially in components designed for bending stiffness.
- It is a renewable resource, the production requires a small amount of energy, CO_2 is used, and oxygen is given back to the environment.
- It can be produced with low investment at low cost, which makes the material an interesting product for developing countries.
- Thermal recycling is possible, where glass and synthetic fibres cause problems in combustion furnaces.
- It has good thermal and acoustic insulating properties in building applications.

Disadvantages of vegetable fibres are listed here.

- It usually has lower strength properties than human-made fibres, particularly when impact strength is necessary.
- Quality varies depending on unpredictable influences such as weather.
- Moisture absorption occurs, which causes swelling of the fibres.
- Maximum processing temperature is restricted.
- It has lower durability, which can be considerably improved by fibre treatments.
- Price can fluctuate by harvest results or agricultural politics.

Many researchers are working for the mitigation of disadvantages of vegetable fibres and on the modification of their characteristics in order *i* to optimize their performance as reinforcement in composite technology.

Table 19.1 presents the most commercially available vegetable fibres. The selection was made based on physical and mechanical properties, cost, durability in natural wet environment, and production. As they are natural products, the fibres are heterogeneous, so the coefficients of variation in some properties are very high. Only as a comparison, the characteristics of E-glass and polypropylene fibres are included in Table 19.1.

19.3 Manufacturing and processing of raw materials

Treatment is required to turn just-harvested plants into fibres suitable for composite processing. For example, in the case of flax, the first step is retting. It is a controlled retting process to get rid of the pectin that connects the fibre bundles with the wood core of the stem. During harvesting, pretreatments, and processing, the handling plays an important role. Failure spots on the fibres can be induced, which causes a reduction of the tensile strength.

In this section, the manufacturing and processing of vegetable fibres that grow on a large scale and in a large area and that are important in international trade will be discussed.

Table 19.1 Physical and mechanical properties of vegetable, E-glass, and polypropylene fibres

	Properties				
Fibres	**Density (g/cm^3)**	**Tensile strength[a] (MPa)**	**E (GPa)**	**Elongation at failure (%)**	**Water absorption (%)**
Jute (*Corchorus capsularis*)[b]	1.36	400–500	17.4	1.1	250
Coconut (*Cocos nucifera*)[c]	1.17	95–118	2.8[d]	15–51	93.8
Sisal (*Agave sisalana*)[c]	1.27	458	15.2	4	239
Banana (*Musa cavendishii*)[b]	1.3	110–130	–	1.8–3.5	400
Curauá (*Ananas comusus* var. *erectifolius*)[e]	0.92	1250–3000	30–80	4.5–6	–
Bamboo (*Bambusa vulgaris*)[c]	1.16	575	28.8	3.2	145
E-glass[f]	2.5	2500	74	2–5	–
Polypropylene[g]	0.91	850	6	21	–

[a] Tensile strength strongly depends on type of fibre, being a bundle or a single filament.
[b] Rehsi (1988).
[c] Agopyan (1988).
[d] Guimarães (1984).
[e] Fordos (1988).
[f] Ikai et al. (2010).
[g] Satyanarayana et al. (2007).

Data from Savastano Jr., H., Santos, S.F., Agopyan, V., 2009. Sustainability of vegetable fibres in construction. In: Sustainability of Construction Materials, vol. 1. first ed. Woodhead Publishing Limited/CRC Press LLC, Cambridge/New York/Washington DC, pp. 55–78.

19.3.1 Sisal fibres

The sisal (*Agave sisalana*) fibres are easily obtained from the leaves, although these fibres are not the best ones from agave plants. Sisal is produced in South America (eg, Brazil and Venezuela) Africa (eg, Tanzania, Kenya and Madagascar), and Mexico, where it originated. Other Central American countries produce small amounts of this fibre. Production patterns differ between countries. In Tanzania and Kenya, sisal is predominantly a plantation crop, whilst production in Brazil is largely small-scale. In 2013, the annual production of fibre in Brazil was about 150,000 tonnes, making it the largest producer of sisal in the world. Brazilian production is concentrated in the states of Bahia (95.8%), Paraiba (3.5%), Ceará (0.4%), and Rio Grande do Norte (0.3%), all located in the northeast region of the country.

The sisal plant and its products have proved, over centuries of natural and commercial production, that they can serve humankind as a sustainable renewable resource. Sisal has a wide variety of traditional applications such as twine, ropes, string, and yarn, and can also be woven into carpets, mats, and various handicrafts. However, sisal has good potential as reinforcement in polymer (thermoplastics, thermosets, and rubbers) composites due to its low density. One key advantage of these composites is their strength compared to their weight, that is, high specific strength.

The use of sisal composites in automotive components and other furniture is gaining popularity. Residues from sisal extraction can be used for making biogas, pharmaceutical ingredients, and building material (Li et al., 2000).

Today, sisal leaves are also being used for pulping. Sisal biomass contains a high proportion of cellulose, and its pulp is a substitute for wood fibres in the paper industry. There is much research into applying it to the cementitious (Savastano and Warden, 2005; Savastano et al., 2009; Tan et al., 2012; Melo Filho et al., 2013; Santos et al., 2015a) and polymeric (Fung et al., 2003; Chand and Jain, 2005; Vilaplana et al., 2010; Ramzy et al., 2014) matrices as reinforcement.

The procedure of decortication of sisal fibre is very crude. It can be dangerous for the workers if they do not use proper procedures for this operation. Fig. 19.2 shows some production stages in the cordage industry that generate residues of sisal. Thousands of simple machines powered with diesel engines are spread out in the sisal plantations. These machines mechanically separate the fibres from the mucilage, but about 40% of the fibres, the short ones, remain in the mucilage residues. The acidity of the fibres is neutralized simply by washing in water. The fibres are bleached in the sun. During processing, a further 10% of fibres are lost as residues. Therefore, only 3% by weight of the leaves is recovered as long fibres.

The global market for sisal fibres has strongly oscillated since 1993 (Fig. 19.3). Brazil has benefited from China's growing import demand. African sisal is strongly demanded for various nontraditional applications. In Africa, the prices increased from around US$900 per tonnes in early 2010 to around US$1900 through 2014. In Brazil, the price increased from US$700 during 2010 to around US$1600 in the beginning of 2015 (FAO, 2015).

Fig. 19.2 (A) Sun drying of sisal, (B) separation of sisal bundles for cleaning, (C) weighting of sisal, and (D) residues of sisal obtained from the cordage industry.
From Savastano Jr., H., Santos, S.F., Agopyan, V., 2009. Sustainability of vegetable fibres in construction. In: Sustainability of Construction Materials, vol. 1. first ed. Woodhead Publishing Limited/CRC Press LLC, Cambridge/New York/Washington DC, pp. 55–78.

19.3.2 Coconut fibres

Coconut is a tall cylindrical-stalked palm tree, reaching 30 m in height and 60–70 cm in diameter. It is a tropical plant for low altitudes. It needs sunshine and a soil rich in calcium and phosphorus. It is, thus, generally suitable for cultivation in sandy seashore terrains (Agopyan, 1988; FAO, 2002).

The most important part of the tree is its fruit, which is egg-shaped, about 30 cm long, and about 25 cm in diameter. The fruit has a large, central cavity, which contains a sweet liquid (coconut water). The external layer of the fruit is thin and smooth; its fibrous mesocarp is 3–5 cm thick, and the endocarp is very hard. The number of fruits per tree varies, depending strongly on soil conditions (Agopyan, 1988).

Among people in tropical countries with ready access to coconut palms—or to fresh coconuts sold by urban street vendors—coconut water is renowned as a refreshing, highly nourishing drink with a delicate aroma and flavour. It is estimated that 350 million litres of green coconut water are consumed in Brazil every year (Prieto et al., 2011; Pereira et al., 2013). After the consumption of the coconut water, the resulting waste generates significant and increasing volumes of useless material, since 80–85% of the gross weight

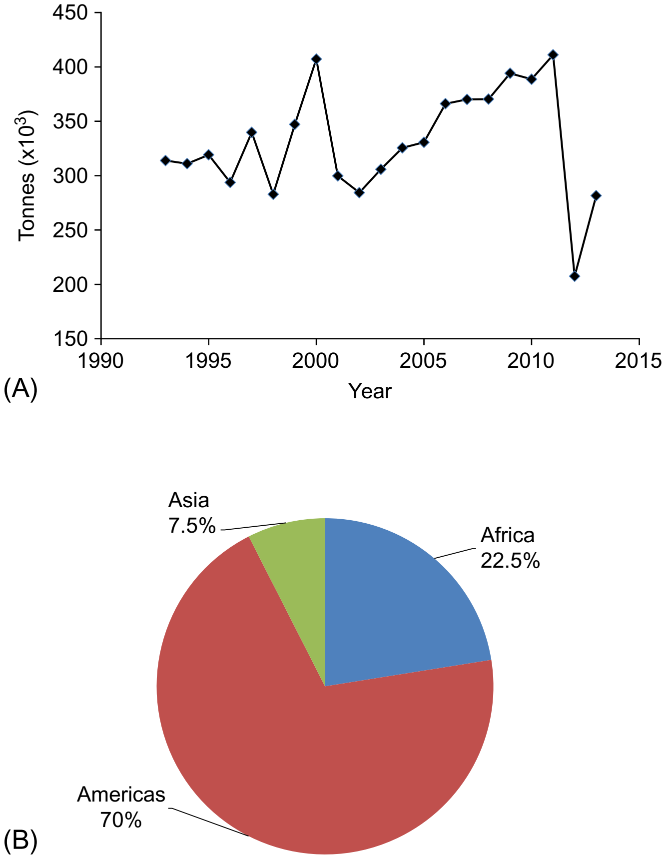


Fig. 19.3 (A) Production of sisal per year and (B) by region (FAO, 2015).

of the green coconut is designated as residues. In Brazilian coastal urban centres, 70% of the waste generated is green coconut husk (Prieto et al., 2011; Pereira et al., 2013).

Although coconut cultivation is concentrated in tropical belts of Asia, it is found in Latin America on a smaller scale (Fig. 19.4). Brazil is the fourth largest producer of coconut in the world, accounting for more than 80% of the coconut production in South America. Brazil produced in 2013 about 2.8 billion tonnes of coconuts (*Cocos nucifera* L.), mainly in the northeast region (FAO, 2015).

Coconut fibres (coir) can be extracted from either mature or immature fruits (Fig. 19.5). The lignocellulose fibres can be obtained from the mesocarp of the coconut fruit, which constitutes about 25% of the nuts. It is one of the least expensive among various natural fibres available in the world. It is not brittle like glass fibre, it is liable for chemical modification, and is nontoxic. But the waste from its disposal causes environmental problems (Tomczak et al., 2007a).

Asasutjarit et al. (2007) conducted a study on the development of coconut-based lightweight cement boards. They were used as building components for energy conservation.

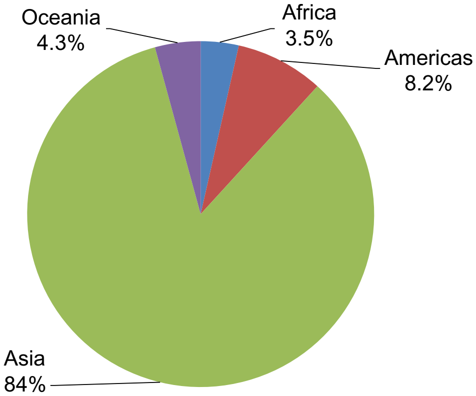


Fig. 19.4 Production of coconut by region (FAO, 2015).

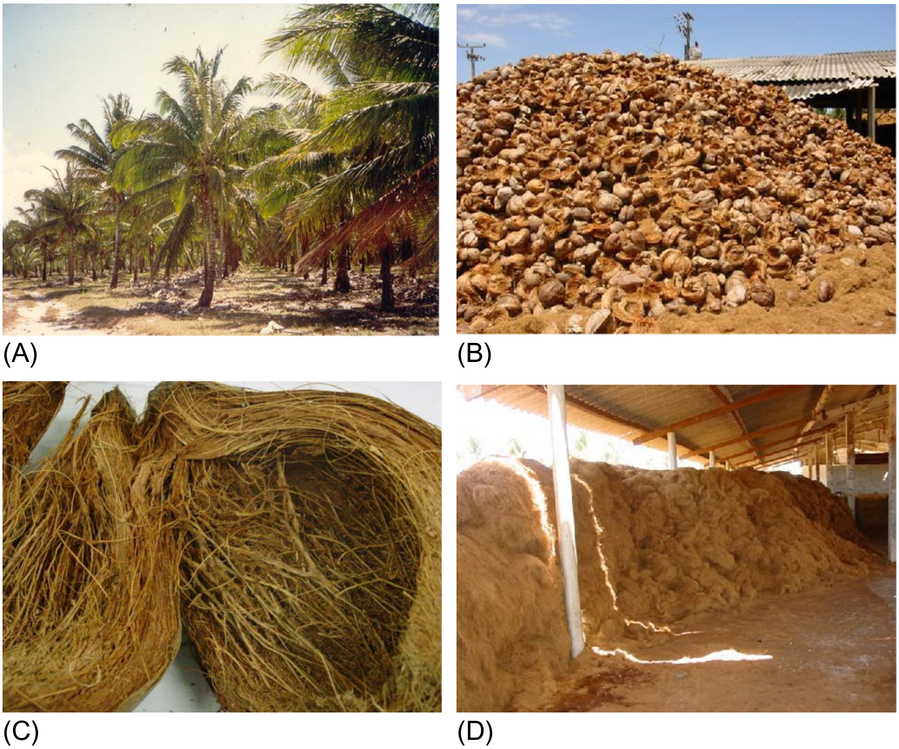


Fig. 19.5 (A) Coconut plantation in northeast region in Brazil, (B) deposits of coconut husk, (C) detail of the fibres in the coconut husk, and (D) extracted coconut fibre.
From Savastano Jr., H., Santos, S.F., Agopyan, V., 2009. Sustainability of vegetable fibres in construction. In: Sustainability of Construction Materials, vol. 1, first ed. Woodhead Publishing Limited/CRC Press LLC, Cambridge/New York/Washington, DC, pp. 55–78.

John et al. (2005) also conducted a comparative study of the microstructure of both new and in-use aged blast-furnace slag cement reinforced with coconut fibre. Aged samples came from internal and external walls of a 12-year-old house built in São Paulo, Brazil, that remained in acceptable conditions after this period under normal utilization. The panels of the house were produced using 1:1.5:0.504 (binder: sand: water, by mass) mortar reinforced with 2% of coconut fibre by volume. After 12 years, the cement was fully carbonated. Fibres removed from the old samples seemed to be undamaged when examined under scanning electron microscopy (SEM). Qualitative determination of lignin content by Wiesner reaction suggests that old samples have lower content of guaiacyl lignin units. Nevertheless, the total lignin content of old fibres when measured by the acetyl bromide method is comparable to that reported in literature. No significant difference was found in the lignin content of fibres removed from external walls and those removed from internal walls.

Pereira et al. (2013) showed that the mechanical performance evaluated by bending test after 28 days reached the modulus of rupture (MOR) of 15.7 MPa after the accelerate ageing, for the composites reinforced with green coconut fibre and with high levels of Portland cement replacement by rice husk ash. This result demonstrates that the use of green coconut fibre can be very promising for the reinforcement of the binary cement-based matrix.

Globally, around 1.2 million tonnes of coconut were produced in 2013, mainly in Asia, in countries such as India and Sri Lanka. India and Sri Lanka are also the main exporters, followed by Thailand, Indonesia, Malaysia, Vietnam, and the Philippines (Fig. 19.6). Around 80% of the coconut produced is exported in the form of raw fibre.

Coconut fibre production is normally rudimentary. Old fashioned equipment crushes the husk and separates the fibres. Some industries have modern equipment that can separate long fibres (with more than 110 mm of length) suitable for brushes and threads.

As a by-product generated in the fabrication of other coconut products, coconut fibre production is largely determined by demand. Abundant quantities of

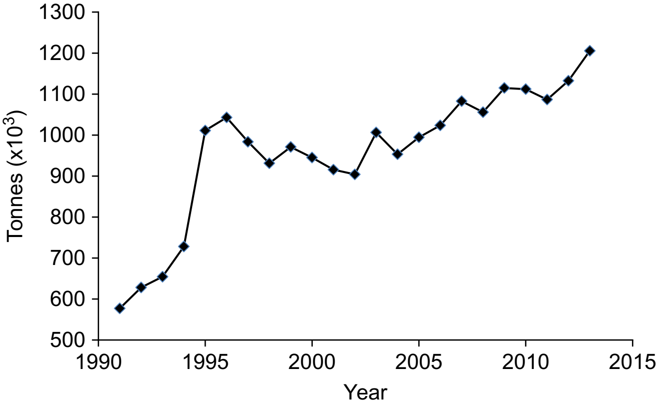


Fig. 19.6 Global production of coconut fibre (FAO, 2015).

coconut husk imply that, given the availability of labour and other inputs, coconut producers can adjust relatively rapidly to market conditions and prices. It is estimated that ~10% of husks is utilized for fibre extraction, satisfying a growing demand for fibre and coconut products. Production of coconut fibre takes place in small- or medium-sized units, mainly in India, Sri Lanka, and Thailand. During the 1990s, production in India expanded by 8.2% annually in order to meet domestic demand, whilst in Sri Lanka, a major exporter of coconut fibre, production contracted due to weakening export and domestic demand. In the medium term, global production increased from an average of 577,000 tonnes in 1991 to 1.2 million tonnes in 2013. Most of the expansion in production is likely to take place in India, with some modest growth in Sri Lanka (FAO, 2015).

Coconut industries in India and Sri Lanka have a strong export orientation with the sale of traditionally made yarn, mats, matting, rugs, and carpets, but substantial technologies to produce better quality and more attractive coconut products with better consumer acceptance have been achieved (Vilaplana et al., 2010; Pereira et al., 2013; Zhang and Hu, 2014).

Therefore, a recent effort to diversify and encourage the further use of coconut fibre has come from a research project implemented within the framework of the FAO Intergovernmental Group on Hard Fibres (IGG/Hard Fibres) and the Common Fund for Commodities. The project aims to develop new technologies to produce building and construction materials from nonretted coconut husks (FAO, 2015).

19.3.3 Bamboo fibres

Bamboo is the most important nonwood forest product and in India it is known as the ‘poor man's timber’. In China, it is the valuable raw material for the booming bamboo industry (FAO, 2005). Throughout wide areas of the world, bamboo plants serve a multitude of purposes. Worldwide, there is a growing interest in the development of bamboo products as sustainable, cost effective, and ecologically responsible alternative construction materials (Sharma et al., 2015). Bamboo fibres are very promising reinforcements for polymer and cement composites production due to their high aspect ratio and strong mechanical performances (Yu et al., 2014; Correia et al., 2014; Wang et al., 2015).

The technological development of humanity was supported in its early stages by natural materials such as bone, wood, and shells. As history advanced, these materials were slowly replaced by synthetic compounds that offered improved performance. Today, scientists and engineers continue to be fascinated by the distinctive qualities of the elegant and complex architectures of natural structures (composites), such as bamboo, which can be lightweight and offer combinations of mechanical properties that often surpass those of their components by orders of magnitude (Wegst et al., 2015).

The bamboo culture in the Americas and Asia is ancient. In these areas, the largest pure and dense bamboo forests and the best and largest number of giant species were found. The natives of these areas also developed the best construction technologies in bamboo houses and bridges and came to be crafted builders. Today, there are many applications of bamboo in different fields of aeronautical, chemical, civil, electrical, hydraulic, nautical, and mechanical engineering (Hidalgo-Lopez, 2003; Lobovikov et al., 2005).

About 75 genera and 1250 species of bamboo occur in different countries of the world. *Bambusa vulgaris* is the best known and most widely used species in Asia. For building, *Guadua angustifolia* Kunth is also applied and is a common plant in Latin America, especially in Colombia, Peru, and Ecuador (Agopyan, 1988; Hidalgo-Lopez, 2003).

About 2.5 billion people in the world depend economically on bamboo, and international trade (import and export) in bamboo amounts to about US$31 million (INBAR, 2015). National and local trade is likely a few times higher. There are other numerous examples of the importance of bamboo for national economies and international trade. However, reliable statistics are still lacking. Most of the economic activities related to bamboo are not recorded officially. They are site specific and highly diverse, and they present challenges for official data collection.

The growing industrial and environmental importance of bamboo requires development of more comprehensive statistics on bamboo resources, utilization, and trade. In 2005, the World Customs Organization approved the FAO proposal to introduce 16 new harmonized system codes, including bamboo pulp, panels, furniture, and shoots. The new codes will have a profound long-term effect on bamboo statistics and for the facilitation of bamboo trade and development (FAO, 2005).

During the last 15–20 years, bamboo has developed as an exceptionally valuable and often superior substitute for wood. Bamboo-based panels and boards (particleboards) are hard and durable and may successfully substitute for the wood products. Bamboo may replace wood in many industrial applications and thereby contribute to the saving and restoration of the world's forests (Biswas et al., 2011; Zhang et al., 2013).

Pulp and paper manufacture from bamboo is expanding (FAO, 2005; Hidalgo-Lopez, 2003). Fig. 19.7 shows some steps in production of bamboo paper.

The length of bamboo fibre is longer than the length of hardwood fibre and this results in a stronger pulp. However, pulping conditions are very similar to eucalyptus. This means that bamboo can be pulped together with hardwood (Stig Andtbacka, 2004).

China is one of the biggest bamboo producers in the world, and bamboo is the second most important fibre material for pulp making, with production of ~1 million tonnes. Bamboo utilization is significant to the development of China's pulp industry (Zhao et al., 2010). Particular interest has been given to the use of fibres obtained from renewable and nonwood vegetable sources in composite materials. Bamboo pulp exhibits a set of important advantages, such as wide availability at relatively low cost, zero carbon footprint, and interesting physical and mechanical properties.

Correia et al. (2014) produced bamboo cellulosic pulp by the organosolv process, evaluating different temperatures and reaction times, and the pulps were analysed aiming their future application for the reinforcement of composites. Bamboo presents advantages such as rapid growth, short-term renewal, and ease of spread that indicate it as a potential raw material for cellulosic pulp production.

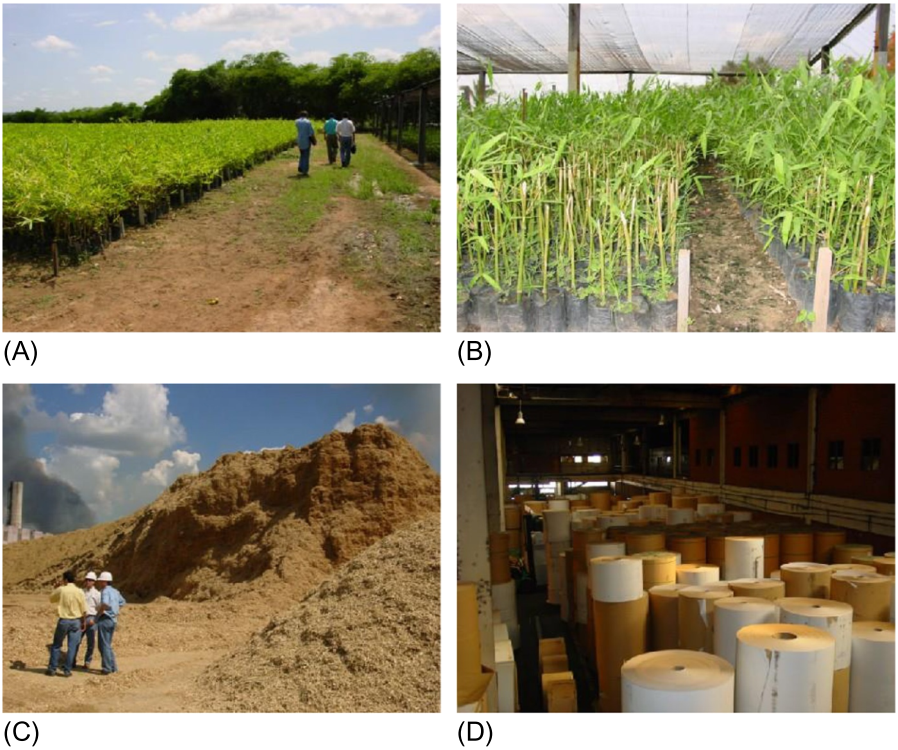


Fig. 19.7 (A) Seedling of bamboo, (B) nursery, (C) minced raw material, and (D) deposit of bamboo paper.
From Savastano Jr., H., Santos, S.F., Agopyan, V., 2009. Sustainability of vegetable fibres in construction. In: Sustainability of Construction Materials, vol. 1, first ed. Woodhead Publishing Limited/CRC Press LLC, Cambridge/New York/Washington, DC, pp. 55–78.

19.3.4 Sugarcane bagasse fibres

Sugarcane (*Saccharum officinarum*) is in the same family as grass, and grows in the form of tall, narrow stalks, or canes (Fig. 19.8). World crop and livestock statistics collected and published by the Food and Agriculture Organization (FAO) of the United Nations are available for years since 1950. According to these data, sugarcane is produced in over 130 countries that are responsible for 97% of the global production.

Developing countries currently account for about 80% of world production (1998–2014). In summary, the global harvest of sugarcane had a nearly twofold increase from 1990 to 2013 (Fig. 19.9A). The annual growth rates are 3.23 and 3.92 of harvested area and production, respectively. Also production is becoming more concentrated among regions and countries (Fig. 19.9B). The 15 top countries, such as Brazil, India, and China among others, accounted for about 70% of the harvested sugarcane area in 2015. World sugar consumption is expanding, reflecting rising incomes and shifts in food and biofuel consumption patterns. Developing countries account for more than 67% of current global sugar consumption—particularly in Asia—and are expected

Fig. 19.8 Sugarcane plantation.
Photo by Loan T. Le; Savastano Jr., H., Santos, S.F., Agopyan, V., 2009. Sustainability of vegetable fibres in construction. In: Sustainability of Construction Materials, vol. 1, first ed. Woodhead Publishing Limited/CRC Press LLC, Cambridge/New York/Washington, DC, pp. 55–78.

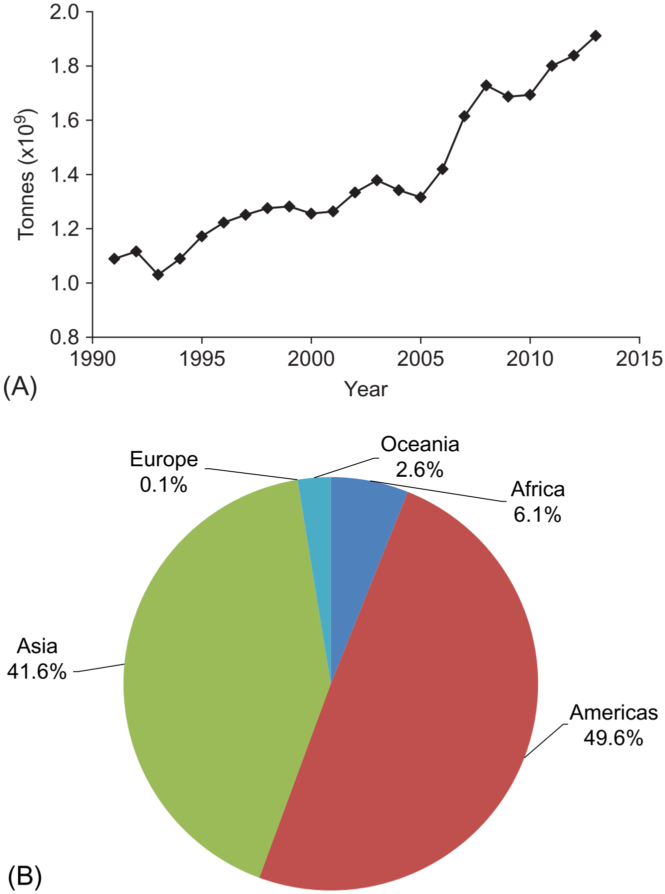


Fig. 19.9 Production of sugarcane (A) by year and (B) by region (FAO, 2015).

to be the primary source of future demand growth. Brazil is the major player and the most competitive supplier in the world sugar market, with the lowest production costs both in field and factory. The country has significantly increased its exports over the last 10 years. The dominance of Brazil in global sugarcane production and expansion derives from its experience and capability to respond to thriving international demand for transport fuels. This demand was recently triggered by measures to decrease greenhouse gas emissions of the rapidly growing transport sector, concerns in developed countries to enhance energy security and become less dependent on petroleum, and the need of many developing countries to reduce import bills for fossil oil (FAO, 2015).

Other applications for sugarcane and its residues (bagasse) are being studied with the objective of generating a sustainable life cycle of production. Sugarcane bagasse, one of the largest cellulosic agroindustrial by-products of the sugar and alcohol industry available in Brazil, is a renewable resource usually used as a biomass fuel for boilers (Fig. 19.10). Sugarcane bagasse is a lignocellulosic fibre residue proceeding from sugarcane culm, obtained after milling it and extracting its juice. The average composition of sugarcane is 65–75% water, 11–18% sugars, 8–14% fibres, and 12–23% soluble solids. The cane basically consists of juice and fibre (Santaella, 2007). The sugarcane bagasse presents the following composition in botanical analysis (by weight): cellulose 41.8%, hemicellulose (as pentosan) 28.0%, and lignin 21.8% (Bilba et al., 2003).

In the future it is possible that the availability of bagasse will increase due to general interest in the biofuels production based on sugarcane.

Limited sources of forests and wood promoted the use of agricultural residues such as sugarcane bagasse. It is a desirable goal from both environmental and economic standpoints. Agrobased particleboards are manufactured from various lignocellulosic materials, usually wooden and mainly in the particle form, which are combined with an adhesive that consolidates under the action of temperature and pressure (Doost-hoseini et al., 2014).

The remaining bagasse is still a source of contamination to the environment, so there is a great interest in exploiting novel applications to sugarcane bagasse fibres. There are successful examples of cement-based materials reinforced with sugarcane

Fig. 19.10 Sugarcane bagasse stockpiled.
Photo by Sérgio F. Santos.

bagasse fibres produced at very low cost and with high potential for buildings in poor areas. In Cuba, the Technical Centre for Construction and Materials, with help of the Cuban Institute for the Research of Sugarcane By-products, has developed sugarcane bagasse–cement panels, similar to those produced from chip wood bonded with cement. Panels of up to 1.20 m^2 have been produced and found useful for construction purposes. Sosa, in several papers, has proposed the use of pressed sugarcane bagasse for the production of panels and sheets (Agopyan, 1988).

Teixeira et al. (2012) evaluated the reinforcement of the sugarcane bagasse fibres in cement-based extruded materials under the effect of accelerated ageing cycles. Fibre-cement composites with 0%, 1.5%, and 5% by volume of sugarcane fibre produced by Auger extrusion process had their performances evaluated, with special interest in evaluation of degradation under accelerated soak and dry tests. The MOR results present similar values between the levels of reinforcement at 28 days of cure and after 200 of soak and dry cycles, but with 28 days of cure the water absorption increased due to the incorporation of large amounts of sugarcane fibre that caused defects in its microstructure and affected the packaging of the composite. The researchers mentioned that further studies should be conducted to optimize the results.

19.3.5 *Curauá fibres*

Amazonic fibres, such as those originated from curauá plant, also have been studied as reinforcement for fibre-cement composites (Satyanarayana et al., 2007; Tomczak et al., 2007b; Amarasekera et al., 2009; Spinacé et al., 2009). Curauá (*Ananas erectifolius*) has been recognized since pre-Columbian days for its valuable fibres. This fibre is one of the unique lignocellulosic fibres of Brazil. Table 19.1 lists some of its characteristics.

SABIC Innovative Plastics has introduced LNP Thermocomp PX07444, a polyamide (PA) 6 thermoplastic compound that is reinforced with up to 20% curauá fibre. Fig. 19.11 compares fibre modulus and specific fibre modulus of vegetable and glass

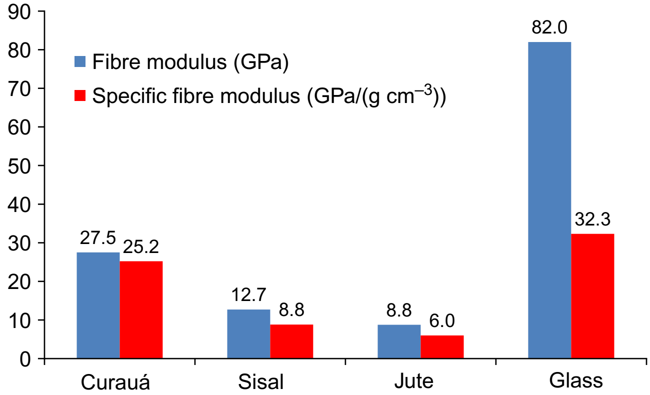


Fig. 19.11 Elastic and specific elastic modulus of vegetable and glass fibres (Amarasekera et al., 2009).

fibres. Curauá fibres have a higher mechanical strength than other natural fibres such as sisal and jute. Besides, curauá fibre has a specific fibre modulus (elastic modulus per density) value near that of glass fibre (Amarasekera et al., 2009).

The use of lignocellulosic fibres as reinforcing agents in polymeric composite materials in substitution of synthetic fibres like glass fibres or carbon fibres is increasing. Previous studies indicate that curauá fibre is a promising material to reinforce thermosets and thermoplastics (Spinacé et al., 2009; Leão et al., 2009; Almeida et al., 2013). According to Tomczak et al. (2007b), demand for this fibre was increasing, with automotive companies alone putting it at about 300 tonnes per annum.

D'Almeida et al. (2010) produced cement-based composites reinforced with 2%, 4%, and 6% short curauá fibres. The results were compared with those found for cement composites reinforced with sisal fibres. Based on the results obtained in the present work, it can be concluded that the use of a cementitious matrix reinforced with curauá fibres, as reinforcement of cement composites, is a promising technique for sustainable materials to be applied in the civil construction industry. However, there is still a lack of technical/scientific information about these important Amazonic fibres. For this reason, the researchers evaluated the main properties of curauá fibres for their future technological application as reinforcement in composites.

19.3.6 Banana fibres

Bananas are the world's most exported fresh fruit both in volume and value (US$7 billion/year) (FAO, 2014). Banana fibre is a bast fibre, which is a waste product of banana cultivation. It is a lignocellulosic fibre, which can be extracted from the pseudostem of the banana plant with better mechanical properties (Srinivasan et al., 2014).

Banana is a perennial crop that grows quickly and can be harvested all year round, and its plants reproduce asexually by shooting suckers from a subterranean stem. The shoots have a vigorous growth and can produce a ready-for-harvest bunch in <1 year. Suckers continue to emerge from a single mat year after year, making bananas a perennial crop.

Bananas fall into two categories:

1. Cooking bananas, including plantains and other subgroups of varieties such as Pisang Awak in Asia;
2. Dessert or sweet bananas, where the Cavendish cultivar is prominent with a 47% share of global banana production. Almost all bananas traded worldwide are Cavendish (Fig. 19.12).

Intensive production on large-scale plantations has increased, and declining producer prices has given rise to many environmental and social challenges. World banana production reached almost 106 million tonnes in 2013, according to Fig. 19.13A. In Fig. 19.13B, see that production is becoming more concentrated in three continents.

In 2012, the volume of global gross banana exports reached a record high of 16.5 million tonnes, 1.1 million tonnes (or 7.3%) above the 2011 level. The increase is primarily explained by the growth of exports from Latin America and the Caribbean from 12.5 to 13.0 million tonnes, despite the poor performance of Ecuador. In Ecuador, the largest banana exporter in the world, banana exports shrank as flooding damaged the

Fig. 19.12 Banana cavendish fruit.
From Savastano Jr., H., Santos, S.F., Agopyan, V., 2009. Sustainability of vegetable fibres in construction. In: Sustainability of Construction Materials, vol. 1, first ed. Woodhead Publishing Limited/CRC Press LLC, Cambridge/New York/Washington, DC, pp. 55–78.

crop, reducing the output by 416,000 tonnes, or 5.6%, relative to 2011. The resulting 410,000-tonnes decline in shipments from Ecuador was more than offset by vigorous exports from other countries, in particular those in Central America (FAO, 2014).

Costa Rica's export quantity reached 2.1 million tonnes in 2012, 126,000 tonnes higher than in 2011. In 2012, banana exports from Colombia dropped by 4.2%, from 1.91 in 2011 to 1.83 million tonnes. In the Caribbean, exports continued to be dominated by the Dominican Republic, which, apart from Belize, is the only country among the Caribbean ACP countries where bananas continue to be a major export despite the preference erosion in the European Union (EU) market, thanks to the strong focus on organic and fair trade bananas. Exports from Asia showed a remarkable recovery in 2012 reaching almost 2.82 million tonnes, well above the previous record of 2.4 million tonnes in 2006. Finally, Africa's exports, which accounted for 3.9% of global banana shipments, grew by 2.4% in 2012 as exports reached 649,000 tonnes (FAO, 2014).

One of the problems in the management of banana crops is waste disposal. Each hectare of banana crops generates around 220 tonnes of plant residues that consists mainly of lignocellulosic fibrous materials. The wastes are usually used as raw material for pulp production and papermaking (Guimarães et al., 2014).

Soffner (2001) has compared two different types of pulping processes applied to wastes from the banana stem, the grain stalk that supports the banana fruits. For banana stem pulping, a CaO process can be considered a technical alternative for pulp production, with delignification rates similar to the NaOH process.

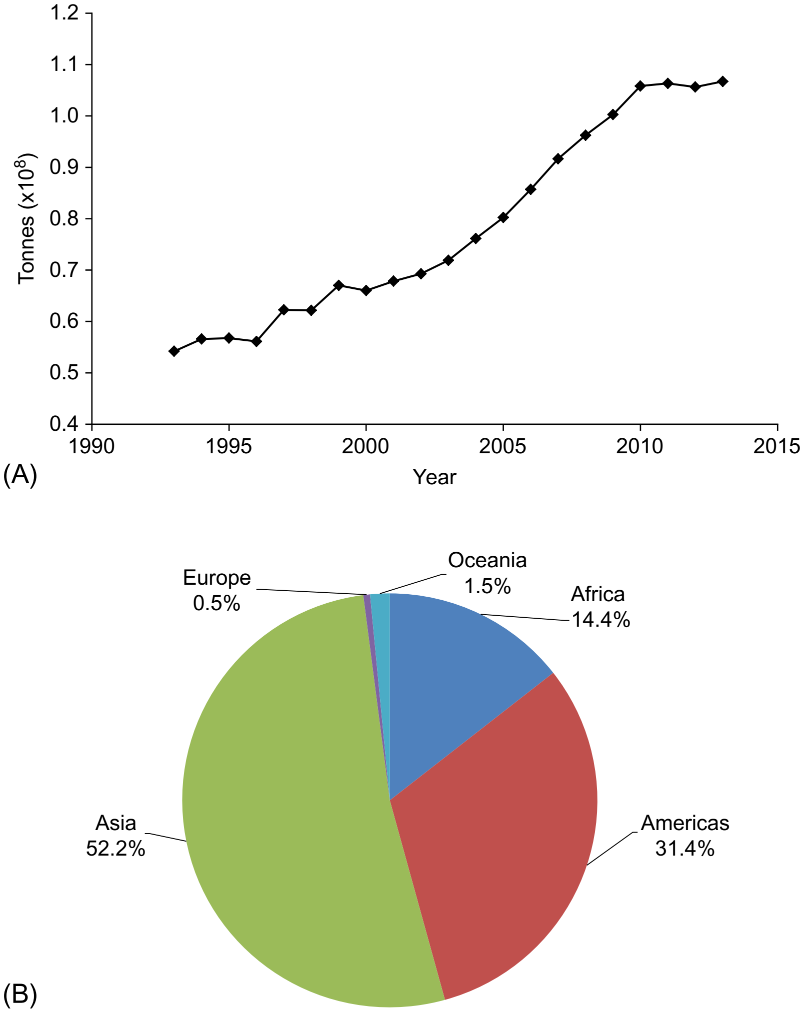


Fig. 19.13 Production of bananas (A) by year and (B) by region (FAO, 2015).

The use of banana pulp and fibres as a reinforcement in cement composites has shown enormous potential in the field of recycled materials and supports their utilization in the sustainable production of building components for civil construction (Savastano and Warden, 2005; Coutts, 1990). Savastano et al. (2009) evaluated fracture resistance (R-curve) and fatigue crack growth of composites of blast furnace slag cement reinforced with pulped fibres of sisal and banana using single-edge notched bend specimens.

Over the past few years, there has been a renewed interest in using banana fibers as reinforcement materials to some extent in the plastics industry. These fibres can be used with human-made or natural polymers to provide a wide range of useful composites in textiles (including geotextiles and nonwovens), particle and other boards, chemical and thermosetting polymer containing goods. Those are applications of increasing interest, as in the future all biocomposites have to be recyclable and fully biodegradable (Kozlowski, 2000; Venkateshwaran et al., 2012; Srinivasan et al., 2014).

Guimarães et al. (2014) verified the feasibility of applying different chemical modifications in banana pseudostem particles as strategy for improvement of the performance of the ensuing particleboards. However, the work of these researchers indicated that is still necessary to improve the treatment variables with NaOH, acetone and water, such as the immersion time, content, and type of chemicals used to adequate the banana tree pseudostem particles for the production of particleboards.

19.3.7 Jute fibres

Jute is a long, soft, shiny vegetable fibre that can be spun into coarse, strong threads. It is produced from plants in the genus *Corchorus*, family Malvaceae (Agopyan, 1988).

Jute is one of the cheapest vegetable fibres and it is second only to cotton in amount produced and variety of uses. Jute fibres are composed primarily of the plant materials cellulose (major component of plant fibre) and lignin (major component of wood fibre). It falls into the bast fibre category (fibre collected from bast or skin of the plant) along with kenaf, industrial hemp, flax (linen), and ramie. The industrial term for jute fibre is raw jute. The fibres are off-white to brown and 1–4 m long.

Jute was growth for many centuries in Bengal before becoming known to the West in the 18th century. Small quantities were imported into England, Europe, and America, but it was only in the 19th century that serious attention was given to jute as a textile fibre (Wood, 1997).

Although jute is not a typical American plant, Brazil is producing it on a large scale, mainly in the Amazon region. The plant grows quite easily in wet and warm areas, and it is harvested 130 days after planting. The average productivity varies from 1500 to 2000 kg/ha (Agopyan, 1988).

The fibres are separated by maceration or decortication. However, for large-scale production only, mechanical decortication is suitable, and this kind of equipment is not available in Latin American countries.

The fibres are used alone or blended with other types of fibres (eg, cotton) to make twine and rope (Sreenath et al., 1996). Jute butts, the coarse ends of the plants, are used to make inexpensive cloth. Conversely, very fine threads of jute can be separated out and made as imitation silk. As jute fibres are also being used to make pulp and paper, and with increasing concern over growing demand for paper adds pressure on the wood pulp, the importance of jute for this purpose may increase. Jute has a long history of use in the sackings, carpets, wrapping fabrics (cotton bale), and construction fabric manufacturing industry.

Jute is predominantly a rain-fed annual crop. Its cultivation is labour intensive, but it requires relatively small quantities of other inputs, such as fertilizer and pesticides, and can be carried out in smallholdings. For all these reasons, jute production is increasingly concentrated in Asia, with market that shows variations over time, mainly in developing countries such as Bangladesh, India, China, and Thailand, which from 1981 to 2013 (Fig. 19.14), together accounted for more than 99.4% of the world production (FAO, 2015).

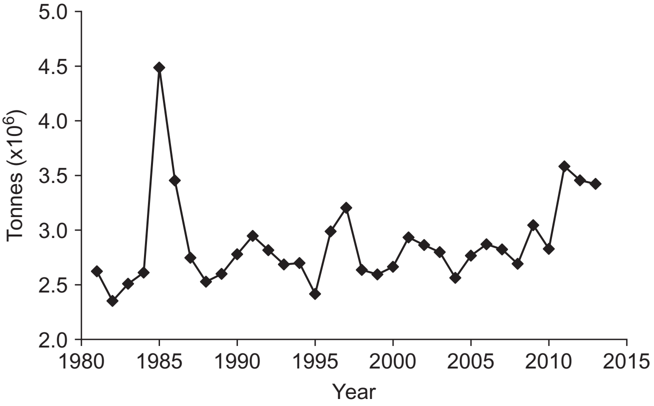


Fig. 19.14 Production of jute (FAO, 2015).

The growing ecological awareness and international environmental regulations have encouraged the investigation of biocomposite materials reinforced with jute fibre. The performance of jute-fibre-reinforced organic and inorganic matrices in strengthening applications has been compared to synthetic-fibre-reinforced composites (Codispoti et al., 2015; Abdellaoui et al., 2015; Arao et al., 2015).

19.4 General uses and market of vegetal fibres

In most developing countries in Africa, Asia, and Latin America, sustainable production of vegetable fibres fulfils a major economical role, which is confirmed by its large contribution to the gross domestic product and to the employment rate.

When determining the properties of vegetable fibres, it is advisable to keep in mind that one is dealing with natural products with properties that are strongly influenced by their growing environment. Temperature, humidity, the composition of the soil and of the air, all these variables affect the height of the plant, the mechanical strength, and density of its fibres. Also the way the plants are harvested and processed results in a variation of properties. For this reason, it is very difficult to use vegetable fibres, but due to low costs they are being largely applied in many areas of the economy.

Today, vegetable fibres form an interesting alternative for the synthetic fibres that are the most widely applied in the composite technology. The use of fibres like coconut, curauá, jute, or sisal in this industry so far is small, since availability of a durable, semifinished product with constant quality is often a problem. Recent research and development have shown that these aspects can be improved considerably. Knowing that vegetable fibres are cheap and have a better stiffness per weight than glass, which results in lighter components, the growing interest in vegetable fibres is clear.

The environmental impact is smaller since the vegetable fibre can come from a renewable resource. The main drawback of using hydrophilic vegetable fibres as reinforcement in polymer composites is the lack of adhesion with most common thermoplastic matrices that have an intrinsic hydrophobic character. Several methods have been applied to overcome this inconvenience. Typically, physical and chemical modifications of fibres and matrices have been performed in order to obtain modified surface properties of the composite constituents.

Hemp, sisal, jute, and flax are the fibres most commonly used to reinforce polymers such as polyolefin, polystyrene, epoxy resin, and unsaturated polyester (Yuanjian and Isaac, 2007; Arbelaiz et al., 2005; Doan et al., 2007; Bourmaud and Baley, 2007). Most of the present applications are in the automotive sector and include composite parts produced by thermoforming or thermocompression moulding techniques. The vegetable fibres are in the form of mats and the matrix is a thermoset or thermoplastic polymer. Recent developments on natural fibre-reinforced composites point—for economic reasons—towards the use of the more versatile and faster injection-moulding techniques. The main disadvantage associated with processing through extrusion/injection-moulding is the drastic decrease of fibre length (caused by the high mixing energy applied) and consequent reduction of reinforcing effect.

In several industrialized and developing countries, cellulose fibres derived from hardwood or softwood, are used for the production of cement composites by adaptation of the former asbestos-cement production processes. Asbestos fibre-cement still represents around 80% of the 250 million m^2 of cement-based composites produced yearly in Central and South America, mostly as corrugated roofing elements. In the case of developing countries, there is an enormous need for houses, schools, hospitals, and public service buildings. Therefore, even in periods of economic difficulties, there is a major need for the application of these vegetable fibres to accelerate the composites production with adequate performance.

The vegetable fibres are low cost (Table 19.2). When it takes into account the variety and abundance of vegetable fibres, with adequate length, availability in developing regions, including residues, such as those from agroindustries, there is a challenge to the application of these fibres to civil construction. However, it is necessary to improve their durability in the composites (Agopyan, 1988; Mohr et al., 2004).

Changes in the fibre and fibre/cement interfacial region due to environmental interactions can affect the long-term performance of cement-based composites reinforced

Table 19.2 **Average international prices for E-glass and vegetable fibres**

Fibre	Price/tonne (US$), raw
E-Glass	2000
Flax	645[a]
Hemp	70–113[a]
Jute	400[a]
Ramie	1521[a]
Coir	335[a]
Sisal	1308[a]

[a] FAO (2015).

with natural fibres. A significant mechanism of change in composite properties is pulp-fibre degradation as a result of environmental interactions or changes in the fibre itself due to its presence in the strongly alkaline matrix (Mohr et al., 2004).

As shown by researchers, there is a considerable range of short-length fibre residues without use for textile or cordage industries but still adequate for the reinforcement of composites. These researchers proposed the following steps for the use of these residues based on the information collected in technical visits: (a) general identification of the agricultural production; (b) identification of residues that includes the correlation with main products and production processes; (c) available amount of residues and other possible uses with actual demands; (d) local availability and requirements for transportation or processing; (e) market value of the residue; and (f) physical and mechanical properties of composites and components.

19.5 Case study I: use of colloidal silica in cement-based composites

In this case study, the use of commercial colloidal silica as a high pozzolanic product with high dispersion and specific surface area and its effects on mechanical behaviour of the cement-based composite was evaluated, according to previous work (Santos et al., 2014).

19.5.1 Raw materials

Mix design (Table 19.3) was established based on commercial formulations used in the fibre-cement industry in Brazil. Portland cement type CP V-ARI (correspondent to ASTM C150/C150M, Type I) was chosen because the finer particle size and higher reactivity. Additionally, this type of cement contains higher levels of tricalcium silicate (C_3S) and dicalcium silicate (C_2S) for the formation of C–S–H.

19.5.2 Composite preparation

For preparation of fibre-cement, *Eucalyptus* cellulosic pulp produced by Fibria Celulose S.A. was previously dispersed in water, with or without colloidal silica, by mechanical stirring at 3200 rpm for 5 min. It was added 0%, 1.5%, 3%, 5%, or 10%

Table 19.3 Mix design of the fibre-cement composites reinforced with *Eucalyptus* pulp

Raw material	Content (% by mass)
Unrefined unbleached *Eucalyptus* pulp (CSF 664 mL)[a]	1.6
Ordinary Portland cement (CP V-ARI)	73.0
Limestone filler	25.4

[a] The Canadian Standard Freeness (CSF) test is a widely recognized standard measure of the drainage properties of pulp suspensions and it relates well to the initial drainage rate of the wet pulp pad during the dewatering process.

w/w of colloidal silica by total mass of cement of the mixture. Ordinary Portland cement (OPC), limestone filler, and cellulosic pulp (Table 19.3) were added forming a mixture with ~20% of solids and stirred at 1000 rpm for additional 5 min.

Fibre-reinforced cement pads measuring 200 × 200 × 15 mm were cast in the laboratory using a slurry-dewatering process followed by a pressing technique, as an approximate reproduction of the Hatschek process applied by the industrial production of fibre cement. The pad of each formulation was pressed at 3.2 MPa for 5 min, then sealed in a plastic bag to keep it wet, held at room temperature for 2 days, and subsequently immersed in water saturated of cement for 26 days with the goal of completing most of the hydration process.

19.5.3 Test methods

Mechanical tests in equilibrium with the air humidity of the fibre-cement were performed using a servohydraulic mechanical testing machine MTS of the 810 series. The prismatic specimens were prepared with nominal dimensions of $16 \times 12 \times 80\,\text{mm}^3$ for mechanical tests of the fibre-cement composite. The MOR was performed with three-point bending configuration. It used a cross-head speed of 5 mm/min. To calculate the MOR, Eq. (19.1) was used:

$$\text{MOR} = \frac{3}{2} \cdot \frac{P_{\max} \cdot S}{b \cdot w^2} \tag{19.1}$$

where $P_{\max}$ is the maximum load reached in the test, S is the span of 64 mm, and b and w are, respectively, width and height of the specimens.

Fracture toughness, K_{Ic}, which evaluates the initial crack growth resistance, was used to characterize the fibre-cement. The specimens of SENB-type (single-edge notch bend) were notched to establish the critical defect size and catastrophic fracture. The configuration used was three-point bending. The prismatic specimens were prepared with a centre plan notch with depth equal to 10% of specimen height and notch tip in 'V' with angle of about of 30 degrees using a diamond saw 0.5 mm thick to simulate a sharp crack.

The values of the maximum load, $P_{\max}$, from load–displacement curves were applied in the calculation of the value of K_{Ic} with Eq. (19.2):

$$K_{IC} = \frac{p_{\max}}{b \cdot w^{1/2}} \cdot y(\alpha) \tag{19.2}$$

where $y(\alpha)$ is a geometric factor. The ratio $\alpha = a/w$ of initial notch length to specimen height was between 0 and 1. The factor $y(\alpha)$ is written as:

$$y(\alpha) = \frac{S}{w} \cdot \left[\frac{3\alpha^{1/2}}{2(1-\alpha)^{3/2}} \right] \cdot \left[1.99 - 1.33\alpha - \left(3.49 - 0.68\alpha + 1.35\alpha^2\right) \cdot \frac{\alpha(1-\alpha)}{(1+\alpha)^2} \right] \tag{19.3}$$

in which S is the span of 64 mm and α is the relative length of the notch, which, in turn, is the ratio of the original length of the notch, a_0, and the height of the specimen, w.

The fracture energy testing was performed using SENB and three-point bending configuration. The specimens with a centre plan notch with 30% of specimen height and notch tip in ‘V’ with angle about of 30 degrees were prepared using diamond saw of 0.5 mm thick. The span was of 64 mm. It was used cross-head speed of 20 µm/min, test control by actuator displacement, to guarantee stable growth of the crack.

For calculation of the fracture energy, γ_{WoF}, the work done by the machine to propagate completely the crack along the specimen was divided by the projected area of the fracture surface (cross section of the specimen), multiplied by two. The integration of the force-displacement curve was made up to the point where the force decreases to 5% of its maximum reached in the test. Besides, a mechanical parameter was obtained to analyse the fracture process from the load–displacement curves and the fracture energy tests called ‘relative crack-propagation work’ to carefully evaluate the mechanical behaviour (Fig. 19.15). This mechanical parameter is obtained by dividing the work of crack propagation by the elastic work. The elastic work is that performed from zero load up to the point of the maximum load. This ratio, thus defined, considers all the work in the effective crack propagation in relation to solely elastic work. Therefore, the higher this relative work, the more resistant the material would be to the propagation of a crack.

19.5.4 Mechanical and physical properties

The content of colloidal silica affected the mechanical behaviour of the fibre-cement. The MOR decreases with increasing colloidal silica suspension content (Fig. 19.16). In fibre-cement, MOR depends on mechanical parameters such as postcracking resistance and ultimate flexural strength. Both mechanical parameters depend on the matrix and fibre–matrix interface, respectively.

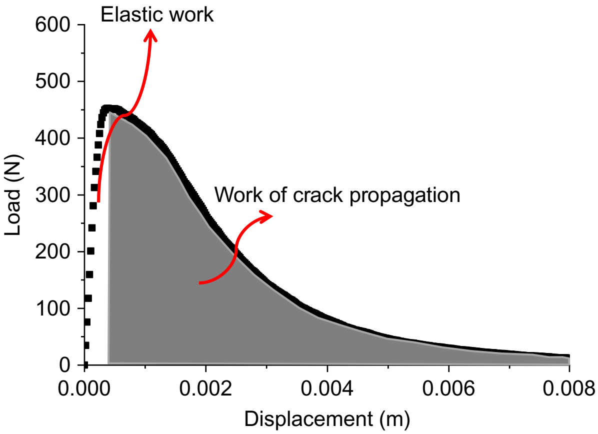


Fig. 19.15 Typical load–displacement curve divided in two regions: elastic work and work of crack propagation.

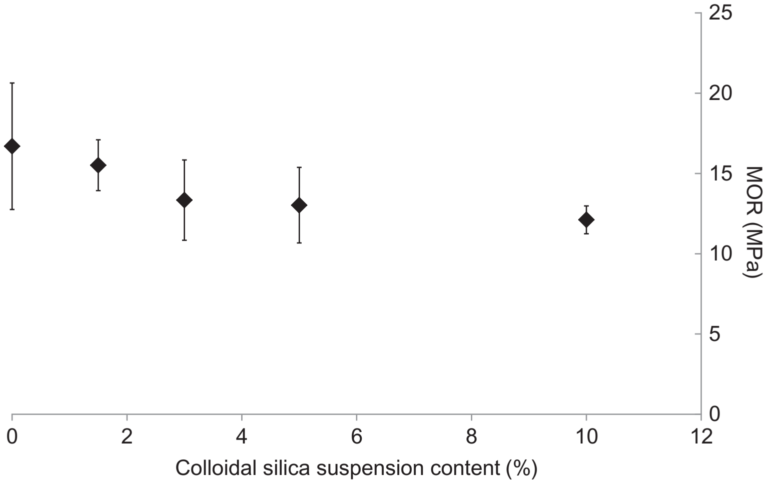


Fig. 19.16 Average values and standard deviations of the modulus of rupture (MOR) of the fibre-cement with different colloidal silica suspension contents.

The influence of colloidal silica on mechanical behaviour of the fibre-cement was also determined by the fracture toughness (K_{Ic}) and energy of fracture (γ_{WoF}). Fig. 19.17 shows the average values of K_{Ic} of the fibre-cement as a function of the colloidal suspension content. The fibre-cement within the range of 1.5–3% w/w of colloidal silica suspension tends to require a higher critical stress intensity factor to initiate crack propagation. The measurement of the energy of fracture (γ_{WoF}) shows indirectly that there is a positive change in the fibre–matrix interface with increasing the content of colloidal silica suspension up to 3% (Fig. 19.18). Formulations with higher contents

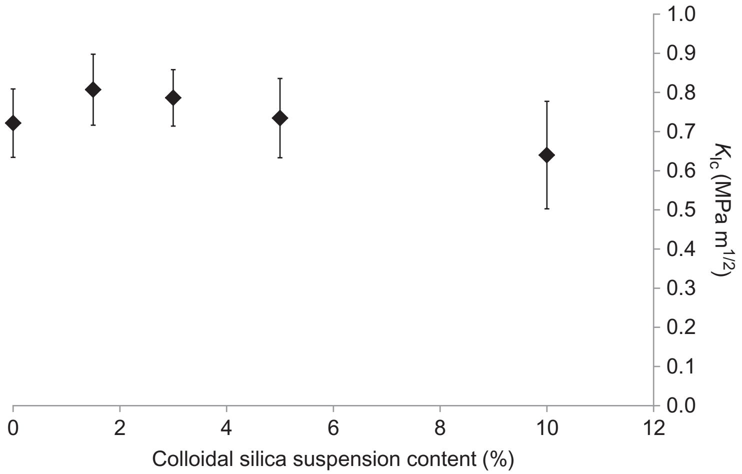


Fig. 19.17 Average values and standard deviations of the fracture toughness (K_{Ic}) of the fibre-cement with different colloidal silica suspension contents.

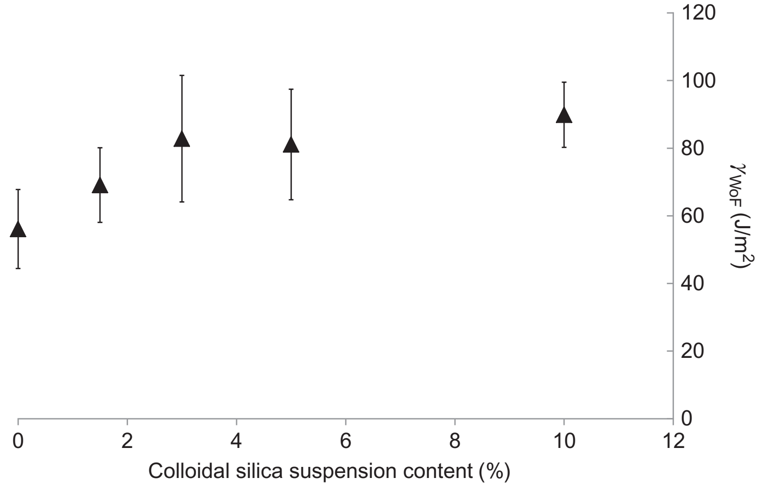


Fig. 19.18 Average values and standard deviations of the energy of fracture (γ_{WoF}) of the fibre-cement with different colloidal silica suspension contents.

of colloidal silica are associated to a stabilization of the γ_{WoF}-values. The significant increase in the average value of γ_{WoF} suggests toughening mechanisms in the fracture process such as pullout.

The relative crack-propagation work of the fibre-cement increases up to 3% w/w of colloidal silica suspension (Fig. 19.19). The 'relative work of crack-propagation' is a mechanical parameter obtained by dividing the work of crack propagation by the initial work. The initial work is that performed from zero load up to the point of the maximum load. This ratio considers all of the work performed for the effective crack

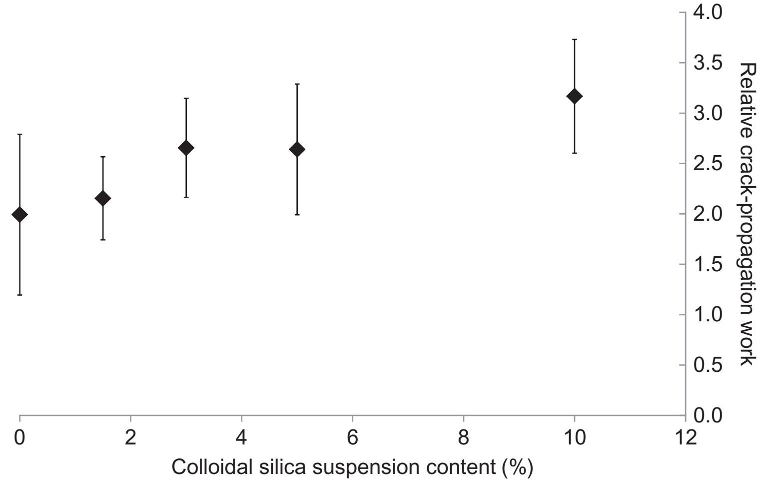


Fig. 19.19 Average values and standard deviations of the relative crack-propagation work of the fibre-cement with different colloidal silica suspension contents.

propagation related to the elastic energy stored in the system. Therefore, a higher value of this relative work indicates that the material is more resistant to propagation of a crack.

The content of colloidal silica affected the mechanical behaviour of the fibre-cement. The MOR decreased with increasing colloidal silica content, but in the range from 1.5% to 3% w/w, it tends to require a higher critical stress intensity factor to initiate crack propagation than other composites presented in the study. The significant increase of 37% and 38%, respectively, in the average value of γ_{WoF} and relative crack-propagation work suggests toughening mechanisms in the fracture process such as pullout.

19.6 Case study II: vegetable fibres in particleboard

The growing demand for materials in the composite wood industries have led researchers to investigate new nonwood sources, such as lignocellulosic biomass (plant fibres and agroindustrial waste) for the production of particleboards (Fig. 19.20).

The present work investigated the feasibility of producing particleboard with densities of 800 kg/cm^3 made with agroindustrial wastes, and the efficiency of two types of adhesives: castor oil polyurethane and urea–formaldehyde.

19.6.1 Raw materials

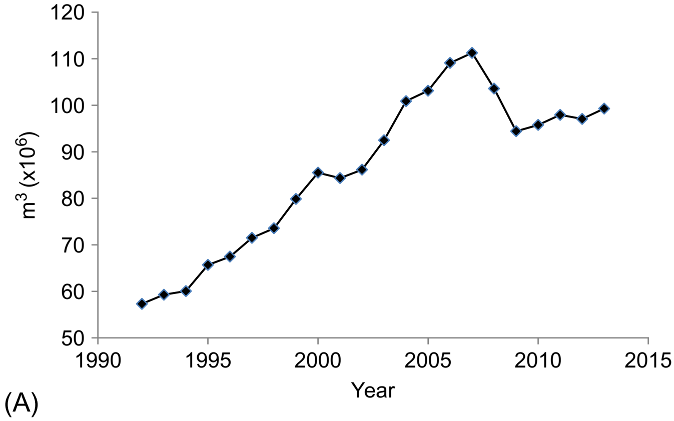
There are several lignocellulosic wastes with potential for use in the production of particleboard, for example, coconut fibre (Fiorelli et al., 2012), peanut shells (Gatani et al., 2013), and wood (Fiorelli et al., 2014).

These panels are generally manufactured from wood particles bonded by synthetic adhesives or other binders, being pressed under heat long enough for the adhesive to cure (Iwakiri et al., 2004). Particleboard can also be produced from any lignocellulosic material that grants them high strength and a predetermined specific weight, since the chemical composition of lignocellulosic materials is similar to that of wood.

Research has evaluated the physical and mechanical properties of particleboard using various adhesives. Urea–formaldehyde and phenol–formaldehyde-based adhesives have noticeable homogeneity (Mendes et al., 2010). However, there is a global trend towards biodegradable, nonpollutant, and renewable products. This trend led to further research, resulting in the discovery and increasing interest in castor oil-based polyurethane adhesives.

19.6.2 Preparation of particleboard

The particleboards were prepared in laboratory using a thermal press, with a loading capacity of 100 tonnes. The process began by collecting the fibres (Fig. 19.21). The waste was dried to a moisture content of ~2–3%, and had an average fibre length of 8 mm and panel density of 800 kg/cm^3. To manufacture the particleboards, 12% of



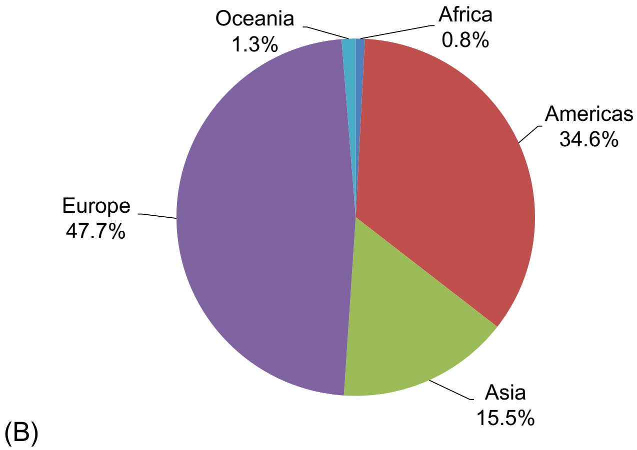


Fig. 19.20 (A) Global production of particleboard until 2013 and (B) production by region (FAO, 2015).

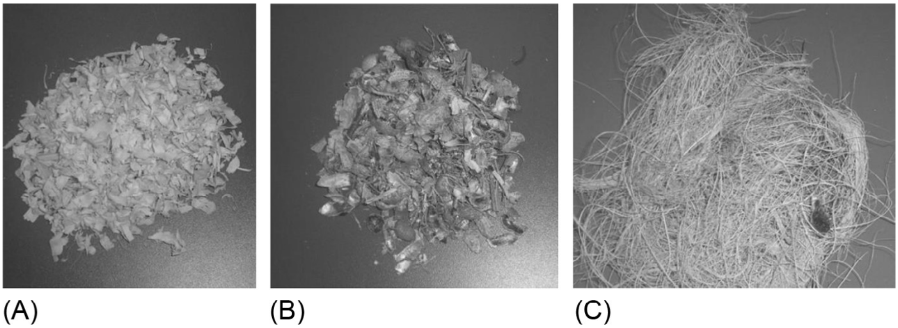


Fig. 19.21 Agroindustrial wastes. (A) *Pinus* sp. wood shavings, (B) peanut shells, and (C) coconut fibre.

the particles mass of castor oil-based bicomponent polyurethane adhesive and urea–formaldehyde were used. More details are presented in Table 19.4.

Fibres were mixed with the adhesive in a planetary mixer (Fig. 19.22A). After mixing, the material was placed into a mould (Fig. 19.22B) and inserted into the hydraulic press (Fig. 19.22C) at pressure of 5 MPa and temperature ranging from 100°C to 140°C. Particleboards with nominal dimensions of $400 \times 400 \times 10\,\text{mm}^3$ thickness were produced. From these particleboards, 10 specimens were taken for each physical and mechanical test.

The values obtained for physical and mechanical properties were subjected to inferential analysis in order to evaluate statistical differences among treatments. A completely randomized design was performed and the results were compared by multiple comparison test (Tukey) if the ANOVA was significant; both tested at $p < 0.05$. The results were evaluated and interpreted using the SISVAR 5.3 statistical analysis software.

Table 19.4 presents the variables to be analysed by the chosen statistical model and the different selected treatments. The columns highlight the agroindustrial waste and adhesives type (castor oil polyurethane, CO and urea–formaldehyde, U) used for each studied particleboard. Each treatment used 10 specimens for testing (ABNT, 2006).

19.6.3 Test methods

The performance of the particleboard was evaluated through physical and mechanical tests in accordance with the normative document NBR 14810:2006 (ABNT, 2006) for Plywood Sheets. The properties evaluated were the thickness swelling (TS), MOR,

Table 19.4 Experimental programme

Particleboard	Variable analysed	Treatments	
Wood shaving of *Pinus* sp. (W)	Adhesive	TW–CO	TW–U
Peanut shell (P)	Adhesive	TP–CO	TP–U
Green coconut shell fibre (C)	Adhesive	TC–CO	TC–U

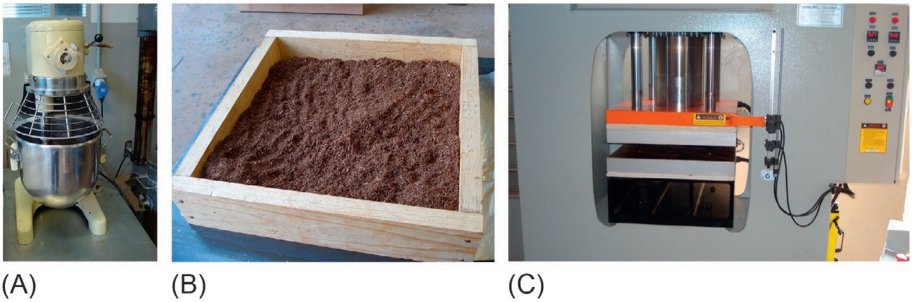


Fig. 19.22 Equipment used in the particleboard production process: (A) planetary mixer, (B) mould, and (C) hot hydraulic press.

modulus of elasticity (MOE), and internal bond (IB) strength. A total of 10 specimens were prepared and tested for each particleboard.

19.6.3.1 Thickness swelling

The TS tests were conducted in accordance with the ABNT NBR 14810:2006 Brazilian Standard. The TS is calculated using the difference in specimen thickness before and after soaking in water for 24 h. Thicknesses were measured using digital callipers with a precision of 0.01 mm. The percentage of TS was calculated using Eq. (19.4).

$$\mathrm{TS}\ (\%) = \left[\frac{T_f - T_i}{T_i}\right] * 100 \tag{19.4}$$

where T_f is the final thickness after soaking for a period of 24 h, and T_i is the initial thickness.

19.6.3.2 Mechanical testing

The IB strength and flexural tests were carried out in a universal testing machine at room temperature, in accordance with the ABNT NBR 14810:2006 Brazilian Standard. The displacement rate during the bonding strength test was controlled at 4 mm/min. MOR, MOE, and specific energy (SE) of eight specimens for each condition were determined by a three-point bending test in an EMIC testing machine (model DL30000), operating with a load cell capacity of 5 kN.

19.6.3.3 Scanning electron microscopy

Samples with dimensions of 15 × 15 mm were prepared to analyse the morphology of the lignocellulosic waste particleboard, as well as the distribution of particles and adhesives.

19.6.4 Mechanical and physical properties

19.6.4.1 Particleboard made of Pinus *sp. wood shavings*

Table 19.7 displays the average values, coefficient of variation (CV), and inferential statistical analysis results for physical–mechanical properties of particleboard produced with *Pinus* sp. wood shavings (TW), using castor oil-based polyurethane adhesive (CO) or urea–formaldehyde adhesive (U).

Particleboard made of *Pinus* sp. wood shavings with castor oil polyurethane adhesive (TW–CO) showed higher average TS values (24 h), when compared with those of samples using the urea–formaldehyde adhesive (TW–U). However, significant statistical difference ($p > 0.05$) was not found for this physical property.

TS tests provide indications related to the particle adhesion and composite porosity conditions. The average TS values at 2 and 24 h listed in Table 19.5 are in agreement with the recommendations of the ANSI A208.1: 1999 Standard (ie, 35%).

Table 19.5 **Physical and mechanical properties of particleboards produced with *Pinus* sp. wood shavings ($d = 800\,kg/m^3$)**

Treatment	Thickness swelling 2 h (%)	Thickness swelling 24 h (%)	MOR (MPa)	MOE (GPa)	IB (MPa)
TW–CO	20.30a	29.31a	14.60a	1.5a	0.54a
CV (%)	13	17	16	12	34
TW–U	14.70b	27.00a	8.70b	1.8a	0.54a
CV (%)	20	15	19	25	13

Values within a given column not followed by the same letter are significantly different ($p > 0.05$).
CV, coefficient of variation.

The TW–CO particleboard presented a higher average MOR value ($p < 0.05$), when compared to that of the TW–U boards. The average MOR value corroborates those obtained by Bertolini et al. (2013), who tested particleboard made with castor oil-based polyurethane and wood particles of different density.

The TW–CO particleboard meets the minimum strength requirements set by ANSI A208.1: 1999 Standard. Therefore, this composite is recommended for applications with medium structural loading.

The IB results listed in Table 19.5 show that no statistical difference was identified ($p < 0.05$) between treatments, proving that the castor oil-based adhesive is an efficient, alternative adhesive for particleboard production and can therefore replace the urea–formaldehyde adhesive. Moreover, the IB values of TW–CO and TW–U particleboards attained the minimum recommendations established by the ABNT 14810:2006 (minimum IB value is 0.4 MPa).

19.6.4.2 Particleboard of peanut shells

Table 19.6 displays the average values, CV, and inferential statistical analysis results for physical–mechanical properties of particleboard produced with peanut shells (TP), using castor oil-based polyurethane adhesive (CO) or urea–formaldehyde adhesive (U).

A statistical difference ($p < 0.05$) was observed for the TS variable at 2 and 24 h between the TP–CO and TP–U treatments, however the TS values after 24 h are lower than the maximum values (35%) recommended by the ANSI A208.1: 1999 Standard.

Fig. 19.23 shows the existence of voids between particles in the peanut shell particleboard. These microstructural elements contributed to increasing the TS value of the composite.

These results are in agreement with those presented by Guler et al. (2008) who studied hybrid particleboard using peanut shells and European black pine (*Pinus nigra*) and made with urea–formaldehyde adhesive.

A statistical difference was observed ($p < 0.05$) for the MOR, MOE, and IB values between the TP–CO and TP–U treatments. The results indicate that the castor oil polyurethane adhesive in the panels promotes better strength and IB. There is a lower CV

Table 19.6 **Physical and mechanical properties of particleboards produced with peanut shells ($d = 800\,\text{kg/m}^3$)**

Treatment	Thickness swelling 2 h (%)	Thickness swelling 24 h (%)	MOR (MPa)	MOE (GPa)	IB (MPa)
TP–CO	8.89a	27.54a	10.98a	1.2a	0.73a
CV (%)	37	28	13	12	20
TP–U	13.25b	23.97b	8.34b	1.7b	0.41b
CV (%)	18	10	18	25	33

Values within a given column not followed by the same letter are significantly different ($p > 0.05$).
CV, coefficient of variation.

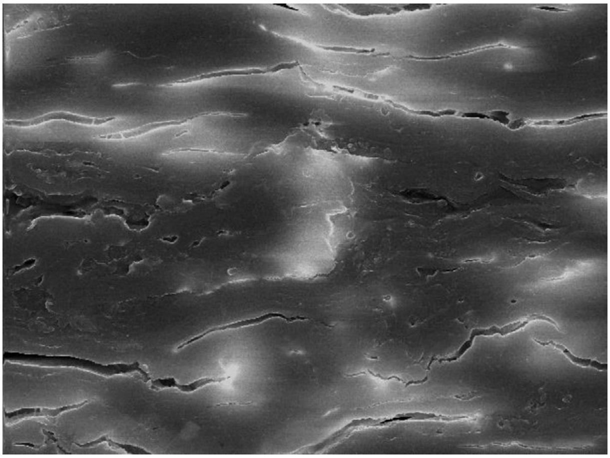


Fig. 19.23 SEM images of particleboard with peanut shells (magnification 500×).

for the mechanical property values of the particleboard produced with this adhesive. Furthermore, the MOR values meet the minimum value recommended by the ANSI A208.1:1999 Standard for low-density particleboard.

Particleboard with peanut shells, TP–CO and TP–U, are restricted to use in non-structural applications and low-moisture environments, as recommended by the ANSI A208.1:1999 Standard.

19.6.4.3 Particleboard with coconut fibre

Table 19.7 displays the average values, CV, and inferential statistical analysis results for physical–mechanical properties of particleboard produced with coconut fibre (TC), and using castor oil polyurethane adhesive (CO) or urea–formaldehyde adhesive (U).

A statistical difference ($p < 0.05$) was noted for all physical and mechanical properties studied between the TC–CO and TC–U treatments. It is worth mentioning that TC–CO particleboard presented the best physical and mechanical properties when compared to the other treatments studied in this work. The TC–U

Table 19.7 Physical and mechanical properties of particleboard produced with green coconut husk fibre ($d = 1000\,\mathrm{kg/m^3}$)

Treatment	Thickness swelling 2 h (%)	Thickness swelling 24 h (%)	MOR (MPa)	MOE (GPa)	IB (MPa)
TC–CO	3.48a	20.95a	18.09a	1.4a	2.25a
CV (%)	116	28	9	12	26
TC–U	25.71b	40.53b	15.76b	2.1b	0.70b
CV (%)	20	13	17	23	22

Values within a given column not followed by the same letter are significantly different ($p > 0.05$).
CV, coefficient of variation.

particleboard presented higher TS values after 24 h, when compared with those of the TC–CO. Additionally, the values are >35% recommended by the ANSI A208.1:1999 Standard.

The TC–CO particleboard has higher MOR value than TC–U particleboard and meets the recommendations established by ANSI A208.1:1999 [9] for high-density particleboard (16.5 MPa). This result indicates that the size and geometry of the green coconut husk fibre, as well as the type of adhesive and compression ratio of the composite, contributed to obtain high-density TC–CO particleboard.

19.6.4.4 Discussion

The performance and behaviour of a particleboard system is dependent on a wide range of variables, such as surface smoothness, presence of extractives, pH of the cellulosic particles, as well as the amount of impurities present, which are related to the environment, such as the level and rate of a change in both temperature and relative humidity (Somani et al., 2003). The bonding mechanism of adhesives is a result of the complex chemistry of the cellulosic substrate, that is, hydrogen bonding with an adhesive, and weak Van der Walls force attraction with others (Somani et al., 2003). Thus, the bonding mechanism is related to the type of binder as well as chemical interaction between agroindustrial particles.

In a chemical structure similar to wood, the adhesion between particles occurs through primary interatomic bonds (ionic, covalent, and metallic) and/or through secondary interatomic bonds (Ramiah and Troughton, 1970). Whilst hydrogen and carbon chains have covalent bonds forming the chemical structure of the polyurethane, urea–formaldehyde adhesive is formed by polyaddition and polycondensation reactions with weak molecular bonds (methylene bridge) (Conner, 2001). Adhesives based on urea–formaldehyde and phenol–formaldehyde are the major adhesives used for bonding wood, but some of these adhesives are very sensitive to hydrolysis and shear stress (Somani et al., 2003).

Castor oil polyurethane adhesive is interesting due to its structural versatility. It presents the particularity of being more compatible with cellulosic particles when compared to other adhesives, due to the possible reaction of hydroxyl groups of the cellulosic particles with the isocyanate groups of the polyurethane (Silva et al., 2006).

Composites used today are characterized by a second strengthening phase that is embedded in a continuous matrix. Particleboard is a composite panel product consisting of cellulosic particles of various sizes that are bonded together with a synthetic adhesive or binder under heat and pressure. In this study, 12% of particle mass of binder was used to produce particleboard with agroindustrial by-products. Thus, the second strengthening phase corresponds to a binder. The load transfer between the matrix (cellulosic particle) and second phase (adhesive/binder) is crucial in determining the strengthening effect.

Fig. 19.24 shows typical stress-specific deformation curves of particleboard with polyurethane adhesive based on castor oil or urea–formaldehyde and reinforced with *Pinus* sp. wood shavings, peanut shells or coconut fibre. It was observed that particleboard with castor oil-based polyurethane presents the best mechanical behaviour, independent of the type of agroindustrial by-products used in this work. The castor oil polyurethane adhesive promoted higher MOR and SE (Table 19.8) values than urea–formaldehyde adhesive.

The improved performance of particleboard made with polyurethane adhesive based on castor oil is attained as a result of the homogeneous distribution of binder in the matrix (cellulosic particles). Subsequently, the stress transfer between cellulosic particles and binder was more effective, in that the MOR values have increased.

SE is related to toughening mechanisms in the particleboard, namely: debonding between cellulosic particles and binder, pullout (extraction of cellulosic particles from

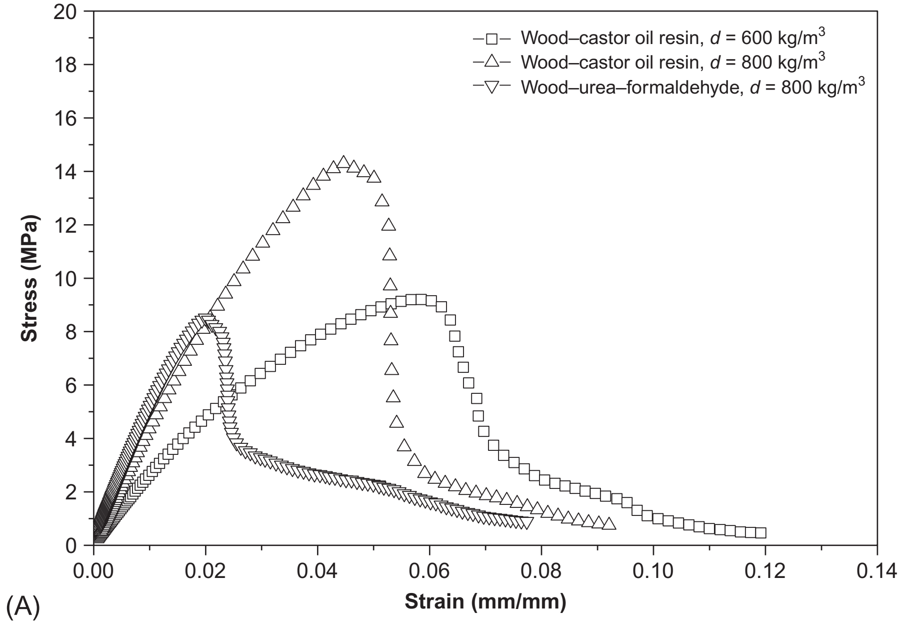


Fig. 19.24 Stress × strain diagram of specific particleboards with castor oil polyurethane adhesive and urea–formaldehyde adhesive: (A) *Pinus* sp. wood shavings,

(Continued)

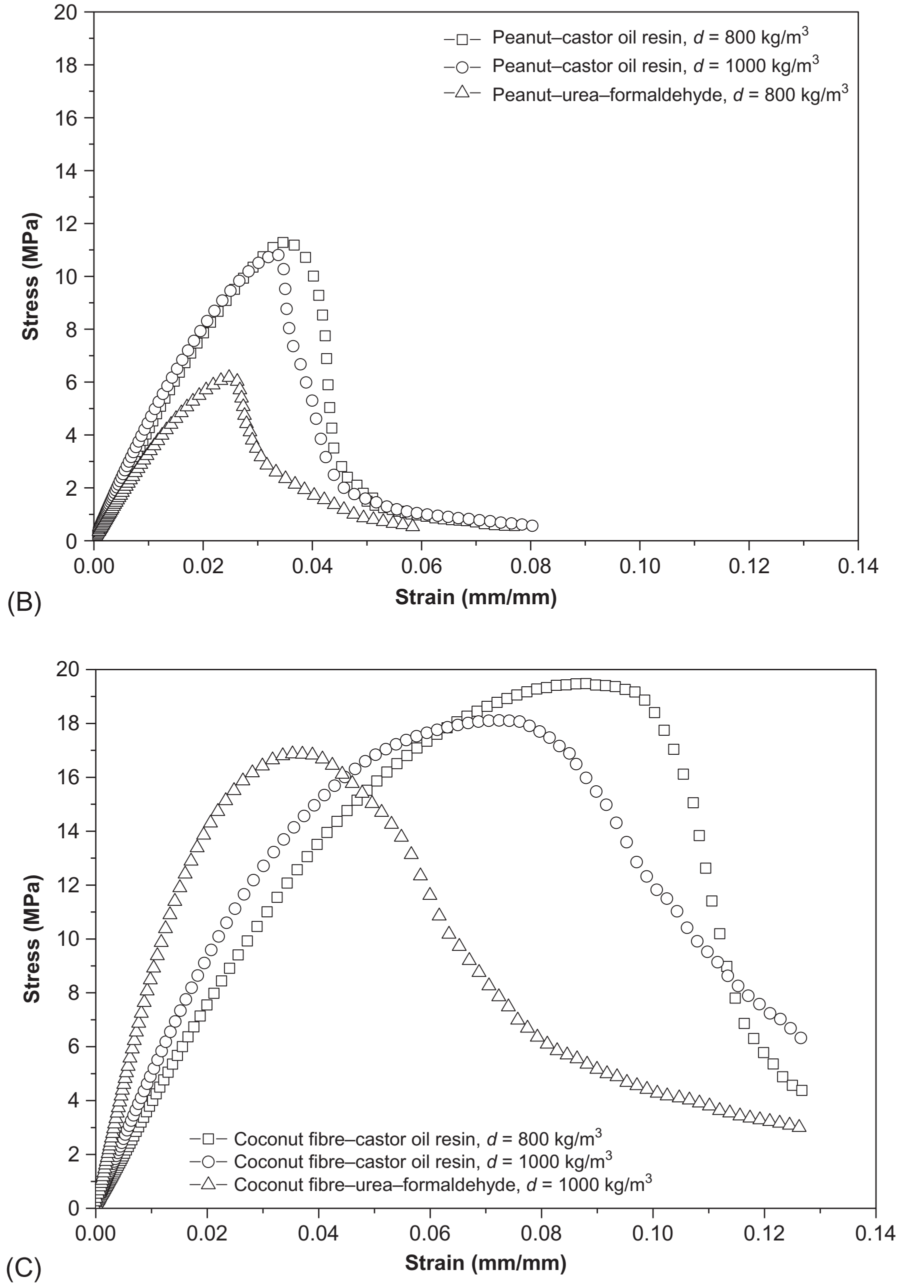


Fig. 19.24 Continued. (B) peanut shells, and (C) coconut fibre.

Table 19.8 Specific energy (SE) and standard deviations for different particleboards

	SE (kJ m^{-2}) Wood *Pinus* sp.		**SE (kJ m^{-2}) Peanut shells**		**SE (kJ m^{-2}) Coconut fibre**	
Density (kg/m^3)	**Castor oil**	**Urea–formaldehyde**	**Castor oil**	**Urea–formaldehyde**	**Castor oil**	**Urea–formaldehyde**
600	3.08 ± 0.61	–	–	–	–	–
800	3.92 ± 0.85	1.61 ± 0.24	2.89 ± 0.28	1.10 ± 0.07	10.20 ± 1.88	–
1000	–	–	2.52 ± 0.40	–	11.16 ± 1.37	9.42 ± 1.09

the binder), and binder-bridging (binder surrounded by cellulosic particles, ie, it is related to IB strength) mechanisms.

Furthermore, the complete elimination of entrapped air (voids) in the binder was not possible, as shown in the SEM image (Fig. 19.23). The voids are formed due mainly to the hydrophilic characteristic of the cellulosic particles and the high reactivity of the polyurethane with moisture. The presence of moisture in the cellulosic particles could serve as an expansion agent in the polyurethane polymerization, because the reaction between water and isocyanate results in the formation of carbon dioxide (CO_2), which is responsible for forming voids in the binder and, consequently, decreasing the cellulosic particles/binder contact area (Milanese et al., 2012). The presence of voids most likely had an influence on the physical and mechanical properties.

The microstructure of an amorphous binder loaded in tension slightly below the glass transition temperature can promote microscopically small, lens-shaped cavities, called crazes. Plastic deformation begins in these regions due to the slight stress concentration caused by these defects. As the material softens, plastic deformation concentrates in this region, resulting in a slight local necking (Milanese et al., 2012). Thus, part of the toughness of the particleboard is attributed to the energy dissipation mechanism through plastic deformation, which occurred in the binders used in this study.

19.7 Future trends

In 2006, the UN General Assembly declared 2009 the International Year of Natural Fibres. Such a decision contributed to Millennium Development Goals by further developing the efficiency and sustainability of these agricultural industries that employ millions of people in some of the world's poorest countries, according to FAO.

The use of vegetable fibres is increasing throughout the world in developed countries, but mostly in developing countries situated in tropical and subtropical climates. Vegetable fibres are used primarily for production of bags, ropes, baskets, and mats, in newer biobased composites, and as sources of papermaking. Vegetable fibres offer several advantages, including their renewable origin, worldwide availability, low cost, low production energy requirements, reduced equipment wear, and biodegradability in some cases, over human-made fibres such as glass, carbon, and aramid.

The use of vegetable fibres as raw material in polymer and cement-based materials not only provides a renewable resource, but also generates a nonfood source of economic development for farming and rural areas and brings new trends in composite materials. However, knowledge, durability, suitable cost-effective design, and fabrication techniques for manufacture should be developed.

The scientific research has demonstrated vegetable fibre to be a useful material in the transformation of recycling waste. Particularly, the use of alternative materials for total or partial replacement of conventional OPC-based composites can be helpful solutions in rural construction. The result can be the production of cost-efficient building elements, with low embodied energy, suitable for developing areas. Potential solutions for housing and rural infrastructure can be achieved throughout the adaptation of already known technologies to figure out the problem of durability as the main drawback of these composites.

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Sustainability of fiber-reinforced polymers (FRPs) as a construction material

20

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20.1 Introduction

The construction industry is one of the leading sectors in economies of all countries worldwide. In Europe, the construction industry with subsectors such as construction products and activities, maintenance and reconstruction works, creates the biggest contribution to Member States' Gross Domestic Product. In the United States, the total value of new private construction in 2012 was around $564 billion. The entire sector accounted for approximately 3.6% of the US gross domestic product in 2014 (http://www.census.gov/const/C30/totsa.pdf).

The materials involved in the construction sector have a wide spectrum ranging from cement to clay, lumber to steel, aluminum to glass. However, in most situations, the sector is very conservative about trying new materials and instead choose traditional ones such as concrete, steel, and lumber since they have a long and proven track record. Although some of these traditional materials have known limitations, they were used long enough to prove their strength and reliability for particular applications. Moreover, the standards related to the construction sector and design and construction guidelines for the stakeholders is well established for these traditional materials, therefore it is not easy to try new and so-called unknown materials such as fiber-reinforced polymer (FRP) composites. Although progress is slow, the use of FRP composites as construction materials is increasing steadily. In some specific applications, such as rehabilitation of existing structures, the FRP composites have gained favor over traditional materials since they provide great advantages such as light weight and ease of application. They also contribute to more durable and hence more sustainable construction components.

20.1.1 Definition of fiber-reinforced polymers/plastics

A FRP composite is a combination of “polymer” and reinforcing “fiber” material. Polymers are organic compounds consisting of long chains of molecules. The basic units of this chain, which repeat themselves, are called monomers. The polymer serves as the binding material for fibers, protects them from environmental effects and mechanical abrasion, and transfers the load from fiber to fiber to provide continuity. Polymers also help regular distribution of the fibers within the cross section. The most

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commonly used polymers for FRPs are polyesters, vinylesters, and epoxies. Each of these polymers has advantages and disadvantages in terms of their cost, application method, and durability characteristics when compared with each other. Therefore, the choice of polymer type for manufacturing the composite material depends on related factors. The other constituent of the FRP composites, the fibers, provide strength and stiffness to the composite material by reinforcing the polymers. The fibers have high strength and stiffness, high aspect ratios for their geometry (very high length to diameter ratio), and uniform cross-sectional properties with low variation in material properties between the individual fibers. The latter property is important to ensure the uniform distribution and transfer of loads through each fiber within any cross section of the FRP composites and prevent premature failures due to stress concentrations caused by irregular geometry or material properties.

FRP composites exhibit superior material properties, which, in most cases, the traditional engineering materials such as metals cannot provide. The FRPs can be manufactured with any length or cross-sectional geometry to obtain the maximum efficiency in terms of the utilization of material strength. Moreover, the FRPs offer lightweight, electrochemical corrosion resistance, durability, and greater efficiency based on their life cycle cost assessment when compared with traditional engineering materials.

20.1.2 Use of fiber-reinforced polymers

Composite materials have been used by human beings for more than 2000 years for advancement of civilization. Natural composites made of timber and plant by-products can be counted as the simplest examples of the composites. FRP composites, which are a type of engineered composites, were used from the early part of the 20th century. Early examples of FRPs were used mainly in aerospace and military applications. With the advancement of materials and methods in recent decades, their usage expanded to many other fields, such as marine and ground transportation, sports, health, energy, and the construction sector.

The first examples of FRP use in building and construction dates back to the 1930s when glass fibers were introduced and the pioneers of engineered materials with glass fibers appeared. In the 1970s, structures with glass FRP components were being constructed at different locations around the world (ie, Benghazi: dome structure; Dubai: airport roof). In the meantime, the polymers of composites and manufacturing techniques continued improving significantly. In the 1990s, an exciting example of a structure with 75% of its materials made of polymers and FRP composites was erected in Finland (Hollaway, 1993). Around the same period in Japan, FRP composites were used as replacements for steel reinforcement to tackle the corrosion problem. In 1988, the first concrete bridge girders prestressed with carbon FRP tendons was constructed in Ishikawa, Japan.

Today, in addition to building construction, the use of FRP is spreading significantly to transportation infrastructure due to its durability and superior material properties. Examples of bridges constructed with FRP reinforcements or prestressing strands are increasing all around the world. However, the rate of penetration to the market is rather slow due to lack of knowledge of precise material characteristics and the limited experience with their long-term behavior. Furthermore, the lack of design and construction standards as well as the higher initial cost compared to traditional construction materials hinder their recognition in the construction sector.

The use of FRPs in the building and construction sector is expected to grow more in the coming decades due to the increase in demand for sustainable and energy-efficient buildings. Identifying the precise limits of material characteristics, improving the manufacturing process efficiencies, reducing cost with alternative production methods, and fully understanding the behavior of composite materials under all possible conditions for short- and long-term applications will surely extend the areas of their applicability and enhance their use in the built environment of modern society.

20.2 FRPs in engineering applications

20.2.1 Types of FRPs and constituent materials

The most commonly used polymer types for FRPs are polyesters, vinylesters, and epoxies. The polyesters are widely used for manufacturing FRP composites. They are the most inexpensive option compared to the two others and their processing is relatively easy. The vinylester polymers are more expensive than polyester polymers, however they provide better resistance to alkaline exposure with their reduced moisture absorption property. Epoxies are the most expensive. They offer excellent bonding characteristics, therefore they are used mostly in wet layup and repair applications.

There are three main fiber types commonly used to manufacture FRP composites: aramid, carbon, and glass. Besides these types, although their use is not yet as common as others, basalt and hybrid fibers also have appeared recently in the FRP composite market. The type of fiber that is used to manufacture the FRP composite for a target application is generally selected based on the desired material properties (strength and stiffness), durability, cost, and availability of the fiber.

Aramid fibers are intermediate- to high-strength fibers commercially known as Kevlar. Aramid fibers have enhanced energy absorption capacity due to their high toughness, which makes them good candidates for impact-resistant materials. They have moderate fatigue resistance and are susceptible to deterioration especially under environmental effects.

Glass fibers are the most commonly used, especially in construction, due to their low cost. Several grades of glass fibers are available for various applications (E-glass, R-glass, and alkaline resistant AR-glass). Although they have high strength, their modulus of elasticity is relatively low, which means they can be used when high deformations are not a significant concern.

Carbon fibers have the highest strength and modulus of elasticity of the all types. They can be manufactured as different grades, starting from standard modulus to ultra-high modulus. They have superior durability properties and excellent fatigue characteristics compared with other types of fibers. Although their mechanical properties are superior in many regards, the cost of the carbon fibers is the highest among all three types.

20.2.2 General material properties

The material characteristics of FRP composites depend on the properties of the constituent materials (material properties of fibers and polymers based on their type), geometry of fibers (cross section of the fibers), fiber volume fraction (ratio of fiber to polymer volume at any material section), and the orientation of the fibers within the polymer matrix. In general, FRP composites are durable (no electrochemical corrosion) and versatile (altered based on needs). They have high strength-to-weight ratio and low thermal conductivity. Their mechanical properties are superior compared with other engineering materials, such as steel, concrete, or timber. As an example, when only considering unidirectional properties, FRP composites are linear elastic with high ultimate tensile strength. Unlike metals, FRPs do not yield and have low strain (high rigidity) at failure. Fig. 20.1 shows the unidirectional stress–strain curves of different types of the FRP composites with respect to the properties of structural steel.

Another advantage of FRP composites is their ability to fit diverse design needs. With a specific combination of selected polymers and fibers, and with assigned fiber orientations, the material properties of the FRP composites can be tailored to meet any particular application requirement.

20.3 Manufacturing process

FRP composites can be manufactured using several different methods including very simple hand placement to automated pultrusion process. Each manufacturing process has advantages and disadvantages based on the method used and desired output. The most common processes are:

- pultrusion
- filament winding
- vacuum infusion

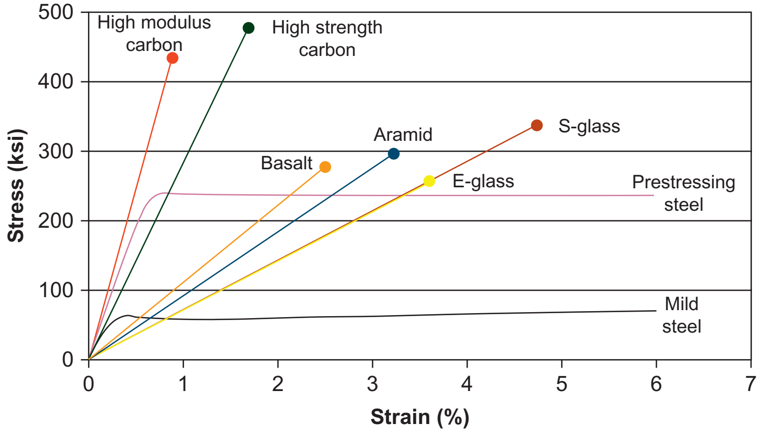

Fig. 20.1 Stress–strain curves of FRPs and steel.

- hand placement
- compression molding
- prepregs

The details of each manufacturing process can be found in literature. In most cases, the manufacturing process is selected taking into consideration the expected properties of the output FRP such as the size and shape, overall cost, functionality, and aesthetic appearance. In addition to factors such as size and length of the FRP composites, the number of elements to be produced in one run also stands as a constraining criteria. It is not practical and cost efficient to produce significant numbers of FRP composite elements by hand placement. On the contrary, for producing several pieces only, a complex manufacturing method such as pultrusion may not be cost effective.

20.4 FRPs in civil engineering

Since their first use in the 1940s, FRP composites and manufacturing techniques have improved considerably, allowing them to be used widely in different fields. One area where the application frequency has significantly increased is the civil engineering field. FRP composites have been successfully implemented in many civil engineering applications including ground improvement applications with piles, soil nails, and anchorages; load-bearing structural members; pressure pipes; tanks and silos; building envelopes; internal and/or external reinforcement of concrete elements; members of transportation infrastructure such as bridge deck components; and all-FRP structures. The use of FRP composites in civil engineering can be classified in two subheadings—geotechnical applications and structural engineering applications. The geotechnical applications appear mostly in the fields of (i) soil improvements such as soil nails and reinforcements, (ii) tunneling and mining applications such as rock bolts and their anchorages, (iii) separation and filtering applications for pavements and roadways with geocomposite sheets and membranes, and (iv) drainage applications with perforated pipes and geocomposites.

20.5 FRPs in building construction and transportation infrastructures

Polymer composites have been used in the construction sector for many years as materials of nonstructural components. At the end of the 20th century, the construction sector was the world's second largest consumer of polymer composites representing 35% of the global market, and this trend has continued to increase (Weaver, 1999). With advances in technology, today FRP composites are also seen as viable and competitive options for load-carrying members of civil structures and structural components of transportation infrastructure. FRPs are now often considered as the first option for rehabilitation and retrofit of existing civil structures, as a replacement for steel in reinforced concrete, and to a lesser extent for new whole-FRP structures.

20.5.1 Nonstructural applications

FRP composites were first used in nonstructural elements at a time when the designers were not confident about the material properties and experience was limited. Today, the main motivations to use composites for nonstructural elements are aesthetic concerns, ease of application, and desire to build sustainable structures. Although many more can be listed, the typical examples of nonstructural composites are fences, window and door frames, pipes and installations, cladding, insulation, decoration, and finishing.

20.5.2 Structural applications

Load-bearing members of civil structures, rehabilitation of existing buildings and bridges, and the structural components of transportation infrastructure are the three main fields for structural application of FRP composites. To construct safe, economical, and functional structures, designers specify that components should have certain stiffness, strength, and durability properties. With the advancement of constituent material performance and manufacturing techniques as well as improvements in defining the material characteristic, FRP composites demonstrate that they are viable alternatives to be used as the main material of load-bearing structural members besides the conventional construction materials. The examples of such applications are steadily increasing all around the world. Transportation infrastructure components such as bridge decks and pavements, poles and pipelines, tanks and silos, onshore and offshore structure components, portable housing alternatives, and energy structures such as wind turbines, the internal reinforcements of all type of concrete or prestressed concrete members, and external reinforcements of rehabilitated existing structural components are several of the many application fields where the FRP composites are emerging (Gangarao and Ruifeng, 2011).

20.5.3 Strengthening/external use of FRP

The engineering community has been in continuous pursuit of new materials to replace traditional construction materials in structural rehabilitation applications. Steel and concrete are used extensively for all types of strengthening application since they are readily available with relatively low cost and well-identified material properties. However, strengthening and maintenance cost of repairs with these traditional materials and their vulnerability to environmental effects, which leads to corrosion and shortened life spans, motivated engineers to look for alternative materials and methods. FRP composites with superior durability and versatility appeared to be viable alternatives for such applications. Their light weight and ease of application provided additional advantages for cases where steel or concrete-based solutions were bulky and impractical.

The most common method of strengthening FRPs is the impregnation of fibers in situ with epoxy resin. This method is called the wet layup method. Since the shape of the FRP composite is formed at the location of application, the wet layup method provides tremendous versatility. However, the method relies heavily on the condition of the application surface for proper bonding and the quality of labor. The latter factor can be controlled by using prefabricated FRP composite pieces (ie, plates, shells, or bars) with well-defined application procedures. To have proper bonding, the application surface should be preconditioned (ie, power washed, sandblasted, etc.) and the sharp corners that may create stress concentrations on FRP composites need to be chamfered so that premature failure is prevented.

20.6 Durability of FRPs

Durability of any material can be described as the condition of maintaining properties close to the initial characteristics for a definite duration. With this general definition in mind, the durability of FRPs can be defined as the ability of composites, with all components that form the composite material (ie, fibers, matrix, or interface), to sustain their material properties without significant deterioration under external effects for a lifetime of interest. A more detailed definition of durability of FRPs can also be found elsewhere (Karbhari et al., 2003).

Starting at the mid 20th century, FRP composites have been used in various fields such as aviation, marine, and construction. Especially in the construction sector, FRP composites have been applied widely in diverse environments including the construction of pipelines, tanks and silos, coastal and port structures, bridge members, internal and external reinforcements for concrete elements, and many more. In all these applications, the FRP composites are exposed to different and challenging environmental and exposure conditions. As an example, the FRP reinforcements for concrete members of the transportation infrastructure function in an alkaline environment and may experience chloride attacks from deicing salts used in the winter, or from sea salt spray at the coastal regions. Although they are resistant to electrochemical corrosion, their material properties may deteriorate in harsh environments or under severe mechanical effects if used improperly or beyond their limits.

Numerous factors that affect durability of FRP composites are reported in the literature. The most significant factors are categorized under two main headings shown in Table 20.1. Although these factors mainly consider the surrounding environmental conditions or imposed loads on FRP composites, one should be mindful that the type

Table 20.1 **Factors that influence the durability of FRPs**

Durability of FRPs	
Environmental effects	Moisture ingress Alkaline exposure Thermal cycles (freeze–thaw) Ultraviolet radiation Fire effects
Mechanical effects	Repeated loads (fatigue) Sustained loads (creep)

of constituent materials and methods of manufacturing are also important and need to be considered when evaluating durability of the composite materials.

20.6.1 Resistance to environmental effects

20.6.1.1 Moisture ingress

In most FRP composite materials, the polymers absorb moisture. The amount of moisture absorption depends on several factors, including:

- type of polymer,
- type and amount of exposed liquid,
- type of fiber,
- fiber to matrix interface conditions,
- actual condition of FRP composite (ie, precracking, sustained load, etc.), and
- other environmental conditions (ie, thermal effects).

Although FRPs are electrochemically resistant, the ingress of moisture can lead to deterioration of the characteristic properties of composites. The mechanism of deterioration due to moisture is still not completely understood, however, the damage to the polymers (softening of polymer matrix) is reported in most cases in the literature (Micelli and Nanni, 2004). The softening of polymer leads to loss of functionality for the matrix and consequently the deterioration of polymer-dependent material properties (ie, flexural and shear strength, and stiffness loss). Moisture durability of FRP-based repairs can be improved by applying appropriate surface preparation techniques and using compatible primers in externally bonded strengthening applications to prevent moisture ingress at interfaces (Dawood and Rizkalla, 2009).

Aramid FRPs absorb moisture and may exhibit strength and stiffness losses. In the case of glass FRPs, the moisture can enter to cross section through interface cracks and may interfere with fibers. This will eventually lead to the loss of strength and stiffness. On the other hand, carbon FRPs exhibit the least deterioration (virtually none) from moisture ingress.

20.6.1.2 Alkaline exposure

Concrete is a typical example of an alkaline environment for internal reinforcement. FRP composites, when used in concrete, are sensitive to alkaline exposure with high pH values. High acidity affects the properties of fibers and the fiber–matrix interface, which consequently leads to deterioration of the overall material characteristics. Polymer matrix is expected to act as a protective layer for fibers against alkaline exposure, however, the degree of protection depends on the type of polymer. Vinylester matrix provides superior protection against alkaline exposure compared to other types of polymers. Although the information related to the deterioration characteristics of aramid and carbon FRP is limited, the stiffness and strength loss of these two types of fibers are relatively low compared to glass fibers. A combination of sustained load or elevated environmental temperature with alkaline exposure creates more severe deterioration for FRP composites, especially for glass FRPs (Mitcelli and Nanni, 2004). Outcomes of further research efforts, especially for understanding the long-term behavior of all types of FRP composites under alkaline exposure with and without additional factors such as various levels of sustained loads and different environmental effects, have become available in the literature in recent years (Shenashen et al., 2014; Chin et al., 2001).

20.6.1.3 Freeze and thaw

FRP composites are affected by low temperatures or thermal cycles of low to high temperature changes. Experimental investigations indicated that the relative elongation/contraction of the fibers and matrix due to their different coefficients of thermal expansion causes residual stresses and consequently creates cracks at the fiber–matrix interfaces. When the FRP composites are exposed to cycles of such repeated thermal changes, the number and the width of the cracks at the interface increase and result in debonding of fiber from the matrix resin. With loss of surface protection provided by the matrix due to debonding, the fibers become susceptible to deterioration caused by other environmental effects (ie, moisture ingress, alkaline exposure). Accordingly, the tensile strength and stiffness of FRP composite is expected to deteriorate under freeze and thaw cycles.

20.6.1.4 Ultraviolet radiation

Extensive exposure to direct sunlight may cause damage to components of FRP composites. The ultraviolet (UV) radiation affects especially the polymers of the FRPs. Among others, aramid FRPs are known as susceptible to UV-imposed damages. Glass and carbon FRPs are reported to be insensitive, therefore do not exhibit any deterioration under direct sunlight. The UV radiation causes discoloration, surface flaws, embrittlement, and cracking at the surface layer of aramid FRP composites. Although these damages are limited to a very thin surface layer, a combined exposure of UV with other environmental effects may cause severe deterioration to durability. This can largely be mitigated by applying UV-resistant coatings to the surfaces of composite elements that are expected to be directly exposed to UV radiation.

20.6.2 Resistance to mechanical effects

20.6.2.1 Fire resistance

It is a known fact that FRP composites lose their integrity under high temperatures due to softening of the polymer matrix. The mechanical and bond properties of FRPs significantly deteriorate after a threshold temperature level is exceeded. This level is called the glass transition temperature (T_g) of the polymer matrix. The matrix starts plasticizing at glass transition temperature and loses its bond strength (Bisby et al., 2005). With the increase in temperature, the organic polymer matrix starts burning and emits toxic gases and heavy smoke, which can be an environmental hazard. On the contrary, the carbon fibers remain insensitive to fire exposure until the temperature reaches 1000°C. The glass and aramid fibers also resist to high temperature above T_g, however they start deteriorating at around 300–600°C, respectively. There is an inclination in the academic and professional community to move toward performance-based design when considering fire resistance of composites. In this case, the specification of FRP systems that could be exposed to fire would be specified based on the desired structural performance under the range of expected fire condition rather than being based on maintaining consistent performance in a standard fire, which can be an extremely severe and unrepresentative test.

20.6.2.2 Fatigue

In engineering literature, fatigue is defined as deterioration of a material or a structural element under repeated load cycles. All materials and products experience fatigue loading throughout their lifetimes. Examples of fatigue loadings include mechanical vibrations, wind, or earthquake-induced dynamic loadings, and repeated loadings such as traffic on roads or thermal effects on any material. Although the amplitude of loading stays below the amplitude of ultimate load that a material can withstand, fatigue loading may lead to failure of the material.

The fatigue behavior of FRP composites under repeated loadings is highly dependent on the properties of matrix material. The polymer matrix cracks after certain number of repeated loadings and accelerates the ingress of moisture and corrosive liquids into matrix/resin interface, which deteriorates the material characteristics of the composite. On the other hand, the fibers are generally insensitive to repeated loading. Carbon fibers demonstrate the best performance under fatigue loading with no loss in strength. Similarly, glass and aramid fibers resist fatigue loading very well with almost no to very small deterioration in their strength.

20.7 Sustainability concept for FRP materials

Maintaining a specific state or a process to produce and to efficiently use a material for an extended period of time can be defined as sustainability. Sustainability, by definition, means using available resources to fill the current needs of societies without violating the right of future generations to have the same or better opportunities. It considers all economic activities, taking into account their environmental impact, and ensures the use of resources in such a way that they can be replaced or renewed. The sustainable use of FRP composites can be specified by understanding the stages of their life span and their impact on the environment based on a detailed assessment of their life cycle.

20.7.1 Life span

The phases of an FRP composite life span are (i) material extraction and production, (ii) manufacturing of the FRPs, (iii) use and reuse, and (iv) end-of-life disposal.

20.7.1.1 Extraction and production of materials

The initial stage of the life span of an FRP composite involves the extraction of constituent materials from their sources as raw materials. Once the raw materials are extracted, they are processed to become input for manufacturing products. The extraction and production process is generally quantified by the amount of energy consumed. A summary of energy intensities needed for extraction and production of selected engineering materials and their constituents is provided in Table 20.2. The input is highly dependent on the extraction and production technology and environment in terms of the infrastructure. Therefore, the energy values vary considerably for each indicated material.

The polymer matrices for FRP composites are produced through a chemical process that demands high energy inputs. Polymer matrices that are commonly used for FRP composites such as polyester or epoxy resins require up to triple the amount of energy for production compared to conventional steel material.

The most common fiber type for FRP composites is glass. The ease of extraction and production of the glass fibers provides the lowest energy intensity. Among all types of fibers, carbon fibers of FRP composites need the most energy input for extraction and production, which leads to higher initial costs. This fact hinders the use of carbon fibers for FRP composites in a widespread manner although they have superior material properties compared to other fibers or other traditional engineering materials.

In general, FRP composite structures are lighter than structures made using most of the other common engineering materials. The light weight increases the versatility. In addition, the light weight reduces the energy input needed for transportation of the FRP composites from their extraction stage to the end of their lives. The carbon emission due to transportation of conventional engineering materials such as steel and concrete is reported to be much higher compared with that for FRP composites.

20.7.1.2 Manufacturing

During the manufacturing stage of FRP composites, a significant amount of energy is needed for processing and curing. The processing involves the integration of fibers and polymer resins with any of the techniques mentioned in Section 20.3. Based on

Table 20.2 Energy input for extraction and production of selected engineering materials (Song et al., 2009)

Material	Energy input (MJ/kg)
Polymers	
Polyester	63–78
Epoxy	76–80
Fibers	
Glass	13–32
Carbon	183–286
Metals	
Steel	30–60
Aluminum	196–257

the technique used, the energy needed for processing varies considerably. The more automated ones (ie, filament winding, pultrusion) tend to demand less energy for manufacturing. Table 20.3 presents the energy input for various techniques. Note that the amounts provided in the table only indicate the energy input needed for manufacturing after extraction and production of the materials.

20.7.1.3 Use and weight reduction

FRP composites are in use in many areas, such as aerospace, marine, automotive, military, construction, and consumer products. With their superior material characteristics, they provide multiple advantages for many applications compared to the use of traditional materials. In the 1970s, only 7% of Concorde airplanes were built with FRP composites. Today, 80% of the Boeing 787 Dreamliner is made of FRP composites. This provides lighter planes and consequently lower energy consumption and carbon emissions. In the 1940s, the first boats were constructed with FRP composites. In recent years, 90% of the hulls of modern boats are made of FRP composites, which provide durability and design flexibility. In the automotive sector, today, 90% or more of the trucks are built with FRP composite bodies. The composite body provides lighter weight and reduces the consumption of fossil fuels and carbon emission. In the military sector, the high resistance of Aramid fibers to impact and abrasion enables the use of these fibers for impact-resistant outfits and body armors.

20.7.1.4 End of life span

FRP composites have extremely long life spans. However, when they reach the end of their service life, they need to be recycled or discarded. Discarding is not favorable due to environmental concerns. On the other hand, chemical recycling or regrinding and reusing are potentially the most suitable end-of-life options. In chemical recycling, the FRP composites are separated to their constituents by chemical means. Although this method of recycling keeps the fibers intact, the fibers are generally not as strong as their initial state, therefore, they are not suitable to be used to produce the same or better quality composite products as their virgin state. This can be considered downgraded recycling. The use of downgraded fibers for new applications is currently a tempting challenge for many researchers, composite manufacturers, and industry partners who use the FRP composites for their production. In the grinding

Table 20.3 Energy input for manufacturing FRP composites (Song et al., 2009)

Manufacturing process	Energy input (MJ/kg)
Vacuum infusion	10
Pultrusion	3.1
Filament winding	2.7
Injection molding	19
Prepreg	40

method, the composites are crumbled into smaller pieces to be used as fillers for other applications. However, the efficiency rate of this method is not better than chemical recycling. Regardless of the recycling method, with current knowledge, the end-of-life processing of FRP composites and its impact on the environment is not well understood. The end-of-life challenges are a major adversity for extended use of FRP composite materials.

20.7.2 Embodied energy

In order to quantify the impact of materials on environment, researchers have been using an energy quantity that is related to the emission of greenhouse gases such as CO_2. The total energy needed for extraction, processing, manufacturing, and transportation of construction materials to construction sites is designated as the embodied energy.

Although embodied energy only quantifies the impact of a construction material at the beginning of its life span (does not consider the use, reuse, or disposal energy), the overall efficiency needs to be considered throughout the life span of a structure. In many situations, a building material or system with higher embodied energy may be considered as a first option because it reduces the total energy requirements of a structure within its service life. In that respect, durable materials with long life spans, such as FRP composites, become a great option with their low embodied energies compared with other construction materials.

The total amount of embodied energy for a typical structure may equal 20% of that particular structure's energy use. Therefore, any reduction in embodied energy reduces considerably the overall environmental impact of the structure.

The embodied energy calculations should be aligned with factors such as climate, availability of materials, and transportation costs. Lightweight construction materials such as FRP composites often have lower embodied energies than heavyweight materials (steel). However, their use may not be efficient if they are not readily available in the surrounding area of the construction site. Similarly, if the construction is expected to experience large thermal fluctuations, a heavyweight construction material with proper insulation may lead to less significant environmental impact for the selected construction due to reduced energy needs for heating or cooling the interior of the structure.

From a sustainability perspective, the embodied energy of any construction material should be considered with respect to the overall durability of the material, the decomposition of its constituents, availability in the surrounding area, use of recycled materials, and waste management. FRP composites have remarkable advantages in several of these aspects, however, their end-of-life processing still remains challenging, which also hinders their sustainable use.

20.7.3 Life cycle cost analysis

Life cycle cost analysis is an assessment method for estimating the performance of materials and services considering their impact on environment, from extraction of raw materials to the end-of-life disposal/recycling stage. It is also known as the

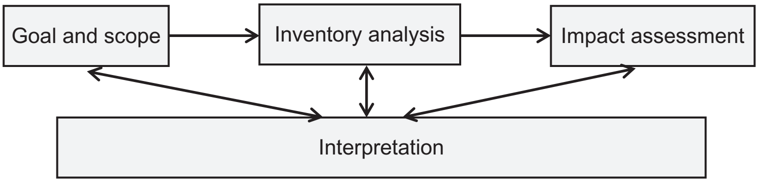

Fig. 20.2 Components of life cycle assessment and their interaction (ISO, 2006).

"cradle-to-grave" analysis since it takes into account all stages of the life span of a material. As it is underlined in ISO 14040 (2006) and ISO 14044 (2006) standards, life cycle assessment methodology considers not only the economic impact (economical input–output methodology) but also the potential environmental impacts of the activities (process-based methodology) throughout the lifetime of the material. The general framework of life cycle assessment methodology is presented in Fig. 20.2. The framework includes four main stages:

(i) *Goal and scope of the study*. The main purpose of the assessment study is stated and the intended audience is identified. The limitations and parameters included/excluded in the study with their justification are also explained.

(ii) *Life cycle inventory*. All the energy and material inputs and outputs are identified and quantified for the system assessed throughout its life span.

(iii) *Impact analysis*. The potential environmental impact of the studied system is evaluated. The life cycle inventory is assessed and converted into environmentally relevant information and the impact analysis is performed based on the compiled inventory and identified system boundaries. It generally includes three sub-steps: classification (sorting of inventory parameters according to the impact categories), characterization (contribution to each impact category), and weighing (total impact over the life cycle).

(iv) *Interpretation*. All the results from the inventory assessment are discussed; the potential impacts are analyzed within the boundaries of the study for the considered system; and conclusions are drawn. If the intention is to conduct comparative analysis, alternative materials or processes can be proposed.

20.8 Recycling of FRPs

Recycling is defined as the combination of four main sequential activities: (i) collection, (ii) processing (categorize and process), (iii) reclamation, and (iv) reuse of recycled materials. FRP composites that are made of polymers and fiber reinforcements have formidable issues in terms of recycling of their constituents, especially at the stage of "processing." FRP composites can be recycled in several different ways including mechanical grinding, incineration, and chemical separation. It is inherently challenging to recycle FRP composites by separating them into their constituent materials without deterioration of the output products. Today, the ratio of recycling for FRP composites is less than 1%. As such, recycling is not emerging as a financially efficient option since the value of the recycled materials is less than the cost of the recycling process. Although, FRP composites demand less energy for production and appear as an economical option compared to conventional construction material based on their life cycle cost analysis, the recycling issue remains a tough challenge to fully comply with the environmental friendly material concept. With the evolution and improvement of new polymer matrices such as biodegradable polymers, FRP composites are expected to be the environment-friendly option among the other materials in the near future.

20.9 Policies and standards for sustainable use of FRPs

In recent decades, with the constant increase in awareness regarding the environmental impact of construction materials and systems, the consumers of the construction sector and society demand more sustainable structures. A timeline for major international events related to political strategies and decisions for sustainability, the evolution of concepts, and the stakeholders' involvement are summarized as follows (CEN, 2009):

- Stockholm, 1972: UN Conference on Human Environment
- 1987, Bruntland Report
- 1992, Rio de Janeiro: UN Conference on Environment and Development
- 1997, Kyoto: Kyoto Protocol on Sustainable Development
- 2000, Lisbon: Lisbon Strategy, 200 Competitiveness
- 2002, Johannesburg: Earth Summit on Sustainable Development
- 2015, Paris: United Nations Summit on Climate Change

Some of the remarkable benefits of sustainable structures include reduced maintenance costs; increased asset value; minimized waste; conserved vital resources such as air, water, and energy; and mitigated environmental impact such as reduced greenhouse gas emissions. In this respect, FRP composites are appearing as cost- and energy-efficient and environment-friendly material choices in sustainable construction applications. A variety of community-driven initiatives or state-regulated requirements for a sustainably built environment have been put into place to guide and inform the stakeholders as well as the consumers of the construction sector all around the world.

20.9.1 Green building initiative (USA)

As a response to the calls of change in US-built environment and to stay competitive in the global and internal market, the concept of sustainability is emerging within the US construction sector with an increasing pace. An important initiative, the Green Building Initiative, with its Leadership in Energy and Environmental Design Program (LEED), is taking the lead for establishing the route for successful implementation of the sustainable structure philosophy. The LEED was developed by the US Green Building Council to set up a framework for implementing sustainable (green) building solutions. It utilizes a rational rating system for the design, construction, and operation of green buildings. Although the implementation of the LEED program is voluntary, the owners, architects, and designers of the structures are contributing to the program to develop and maintain sustainable structures with energy efficiency, low greenhouse gas emissions, and positive environmental impact.

Recognized by LEED as environmental friendly construction materials, FRP composites are now gaining more interest among the stakeholders of the construction sector. FRP composites can now qualify for many credit categories of the detailed rating system of LEED such as Energy Performance and Green Power and Carbon Offsets (LEED, 2013). Besides LEED, a range of other initiatives have been established to achieve sustainable construction in the long run.

20.9.2 Basic work requirement-7 (BWR7) of construction products regulation (EU)

At the beginning of 21st century, the European Union (EU) identified and addressed seven flagship initiatives to boost the smart, inclusive, and sustainable growth of its member states. To achieve sustainable growth, the EU declared that member states need to establish more competitive low-carbon economies with efficient and sustainable use of their resources, protect the natural and built environment by reducing greenhouse gas emissions, and enhance the position of Europe in the global market by developing new green technologies and production methods. Particularly in the construction sector—the key sector in national economies with highest impact on employment and environment—the use of innovative and green materials and systems such as FRP composites plays an important role in fulfilling the aims of the EU.

Several initiatives and instruments on which the EU builds its sustainable construction policy are listed in CEN/TC250 Document N-798 (2009). These instruments and strategies include:

- Integrated Product Policy (IPP)
- Thematic Strategy on the Sustainable Use of Natural Resources
- Thematic Strategy on Waste Prevention and Recycling
- Eco-Management and Audit Scheme (EMAS)
- Ecolabel Scheme
- Environmental Technologies Action Plan (ETAP)
- Green Public Procurement (GPP)
- Eco-design of Energy Using Products Directive (EuP)
- European Compliance Assistance Program—Environment & SMEs

In order to provide a common technical language and offer uniform assessment methods of the performance of construction products, the EU published the Construction Products Regulation in 2011 (EU CPR—305/2011). The Construction Products Regulation, which defines European standards for construction products in view of safety, stability, energy efficiency, presents seven basic requirements for construction works (EU CPR—305/2011). The seventh requirement (BWR7) states that a building should be designed, constructed, and demolished in such a way that natural resources are used sustainably. In this respect, FRP composites provide a unique alternative to traditional engineering materials with their high performance and versatility and low environmental impact, which complies with the requirements of the BWRs.

20.10 Future trends

The academic community is leading advancements in materials development, analysis, and design to enhance the effectiveness of structural composites, increase their sustainability, and provide better guidance for design and installation in structural applications. Such initiatives include investigating alternative, environmentally friendly polymers and fiber types. Since polymers are primarily produced from fossil fuels, recent research has focused on developing plant-based binders that have similar performance characteristics as their fossil-fuel based counterparts. This development would enhance the sustainability of composites broadly by eliminating reliance on fossil fuels. Similarly, some researchers are investigating the use of alternative fibers sourced from natural sources such as plant-based fibers and basalt (a type of volcanic rock). These advances also promise to enhance the sustainability of composites.

Over the past 20 years, researchers have used FRPs for strengthening concrete structures (Belarbi et al., 2012; Murphy et al., 2012; Belarbi and Acun, 2013; Bae et al., 2013). Researchers also began to investigate the use of FRP composites for repair and rehabilitation of existing steel infrastructure as an alternative to replacement. While the use of FRPs in concrete construction has gained widespread use, their use in steel structures is an emerging field. Research has been done on flexural repair (Dawood et al., 2007) and axial repair (Kaya et al., 2015). Other research has focused on combining FRPs with other advanced materials, such as shape memory alloys (SMA), for fatigue repair of cracked steel structures (El-Tahan et al., 2015; El-Tahan and Dawood, 2015).

Further advancements have targeted code developments and adopting a probabilistic approach to developing code equations (Forouzannia et al., 2015). Until recently, due to the lack of rigorous, probabilistic-based design guidance, engineers have traditionally been conservative when designing FRP-composite structural systems. However, by applying probabilistic design approaches and establishing acceptable target probabilities of failure, designers can more efficiently use materials thereby further enhancing the sustainability of structures constructed using FRP composites.

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Sustainability of fibre composite concrete construction

21

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21.1 Introductory overview

The use of fibres in building materials was known at several centuries BC. The ancient Egyptians mixed straw fibres with mud to make their sun-dried bricks. Straw fibres prevented the bricks from cracking. The use of straw for building huts is still practiced in many villages in some of the underdeveloped countries (Mehta and Monteiro, 2004). However, the idea of including any kind of fibres in concrete was not introduced until quite recently. A clue to the idea came from the experiments of Nervi in the early 1940s on the behaviour of slabs of concrete reinforced with closely spaced wire mesh. The spacing of wires was of the order of 12 mm. Nervi observed that the slabs exhibited great flexibility and resilience and that visible cracks did not occur until the steel was stressed nearly to its yield point. Nervi used this material in the construction of long span shells and the hulls of several commercial fishing vessels. However, no sufficient explanation for this apparently unique behaviour was offered at the time (Shah, 1982).

It was not until the early 1960s that the first direct research on fibre reinforcement in concrete was published. In 1960, Goldfein pioneered the use of glass fibre-reinforced plastics in concrete by taking a patent on a method to produce reinforcement in the form of bars made of glass fibres that are bonded together using a resin (Goldfein, 1960). In 1963 and 1964, Romualdi and Batson and Romualdi and Mandel published two papers on fibre reinforcement in concrete (Romualdi and Batson, 1963; Romualdi and Mandel, 1964). The 1963 paper by Romualdi and Batson became the landmark of the contemporary push into research and development of fibre-reinforced construction materials. This was later followed by a couple of papers by Aveston et al. (1971, 1974). Two other reports on fibre reinforcement by Goldfein and Krenchel were published in 1963 and 1964, respectively (Goldfein, 1963; Krenchel, 1964). Soon, interest in developing the new reinforcement technique was spread to engineers and industry. Efforts were directed mainly at finding the best and most economical kinds of fibres to be used as reinforcement. As a result, several kinds of fibres were produced and used for reinforcing concrete.

21.2 Types of fibre composites in the building industry

Two main categories of fibre composites play significant roles in modern construction industry. These are the fibre-reinforced polymers (FRP) and fibre-reinforced cementitious materials used in the concrete industry. The distinction between the two

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lies in the fact that FRP composites are made of fibres and resin and manufactured into the required item in the required shape and size. On the other hand, the fibre-reinforced cementitious materials such as fibre-reinforced concrete are usually made by mixing the fibres with the dry or wet concrete or cementitious binders when the concrete is cast.

The first category has numerous applications in the construction industry and has assumed an increasingly important role replacing or strengthening currently used elements and/or in the repair of structural and nonstructural elements. The subject of FRP composites is very wide and FRP has earned its place as an independent construction material beside timber, steel and concrete. Codes of practice have provided guidelines for the use of such elements (AS, 2008; ACI, 2004, 2006; CSA, 2002; Clark, 1996).

This chapter will focus mainly on cementitious materials reinforced with short randomly distributed fibres and the relation between the developments in this direction and sustainability in concrete construction.

21.3 Fibres used in concrete

21.3.1 Broad categories

The types of fibres used in the concrete industry may generally be grouped as follows:

1. *Organic fibres*: These are either natural or synthetic. Examples of natural organic fibres are cellulose from wood pulp (Morrissey et al., 1985), date palm (Kriker et al., 2008), flax (Boghossian and Wegner, 2008), sisal (Filho and Sanjuán, 1999), banana pseudostem fibres and fibres derived from waste eucalyptus grandis pulp (Savastano et al., 2000).

   Examples of the synthetic organic fibres are polypropylene, nylon and polyethylene (Shah, 1972). These are lighter than other types and inert to chemical attack. They improve the impact resistance of concrete. Therefore, they are used for structures that would receive shocks or explosions. However, they have a poor bond and can reduce the structural strength of concrete due to their lower modulus of elasticity.
2. *Metallic fibres*: Steel fibres are the most common among metallic fibres (Brandt, 2008). These fibres possess high modulus of elasticity, good bond and high elongation at fracture (Wang et al., 2008). Therefore, a great improvement on both compression and flexural strength of concrete can be achieved (Balendran et al., 2002).
3. *Mineral fibres*: Asbestos fibres, glass fibres, carbon fibres and basalt fibres are all mineral fibres.

Asbestos, apart from being expensive, is a serious health hazard as it causes lung cancer. It is no longer used in the building industry and any exposure to it should be avoided (Brandt, 2008).

Glass fibres have been used quite extensively since their introduction in the early 1960s (Biryukovich et al., 1964). The main problem with the commercially available glass fibres (E-glass) is that they are chemically attacked by the highly alkaline environment of cement paste, and as a result they lose their strength with time. E-glass is borosilicate glass and is called E-glass for its high electric resistivity (Bank, 2006). Chemically inert glass fibres were invented by Majumdar (1970) and Majumdar and

Ryder (1968) and are being produced as (alkali resistant) AR-glass. The increase in flexural strength caused by these fibres is about two to four times (Grimer and Ali, 1969). The impact resistance increase is significant although it does not eliminate the concrete brittleness property (Harris et al., 1972).

Another type of mineral fibre is carbon fibres, which are made of filaments that are of 5–10 μm diameter. Thousands of these filaments are wound to make a carbon fibre. Carbon fibre yarn may be used in special fabrics as a textile. The fibres may be used in several applications including composites in carbon-FRP and concrete. These fibres possess very high tensile strength of more than 2000 MPa and may exceed 7000 MPa. Their elastic modulus is commonly in the order of 300 GPa but can exceed 500 GPa. Their density is around 1750 kg/m^3. For fibre-reinforced concrete, carbon fibres whose tensile strength is 4900 MPa and whose tensile E-modulus is 230 GPa were tested (Basche et al., 1999; Yakhlaf et al., 2013; Konig et al., 2001).

21.3.2 Physical properties

Physical properties include geometrical and mechanical characteristics and are important in influencing the behaviour of the fibres and their effect on the properties of the fabricated elements. Geometrical properties are very important and affect the mechanical characteristics of the reinforced concrete (Johnston and Zemp, 1991). The material of the fibre is obviously another decisive factor that determines its behaviour. Table 21.1 summarizes the most relevant physical properties of commonly used fibres.

21.4 Fibres as ingredient of concrete

21.4.1 An overview

The effects of fibre reinforcement on the properties of concrete have been extensively researched (Ramachandran et al., 1981; Naaman and Reinhardt, 1996). Shah summarized the perception of the contribution of fibres in general as being insignificant at the usual application rate of about 1% by volume of concrete and that their role is only confined to the bridging of cracks (Shah, 1991). The addition of polypropylene to concrete has been known to benefit the early concrete life properties and to reduce plastic shrinkage cracking (Carr, 2000). Polypropylene fibres, however, are not known to increase compressive strength (Carr, 1998; Alhozaimy et al., 1996). Alhozaimy et al. have reported neither compressive strength gain nor a benefit to flexural strength as a result of polypropylene addition (Alhozaimy et al., 1996).

Nakagawa et al. reported the effect of nonmetallic fibres on compressive strength (Nakagawa et al., 1989). They found that compressive strength actually decreased with the increase in fibre content. This is quite expected where large volumes of fibres tend to cause entrapped air in the wet mix.

The effects of steel-fibre reinforcement have been the subject of research for many years. Results on the effect of steel fibres on compressive strength ranged from a decrease in compressive strength to an increase as high as 40% (Swamy and Mangat, 1975)

Table 21.1 Physical properties of some fibres used for reinforcing concrete

Fibre	Specific gravity	Diameter (mm)	Length (mm)	Modulus of elasticity (GPa)	Tensile strength (MPa)	Strain at failure (%)
Steel[a]	7.86	0.1–0.6	10–60	200	280–2800	0.5–35
E-glass	2.57[b]	0.0125 (filament)	10–50	72.5	3400	2.5
AR-glass	2.68[c]	0.0125 (filament)	10–50	74	3600	2.5
Carbon (standard)	1.7[b]	0.005–0.01	3–continuous[a]	250	3700	1.2
Carbon (high strength)	1.8[b]	0.005–0.01	3–continuous[a]	250	4800	1.4
Carbon (high modulus)	1.9[b]	0.005–0.01	3–continuous[a]	500	3000	0.5
Carbon (ultrahigh modulus)	2.1[b]	0.005–0.01	3–continuous[a]	800	2400	0.2
Polypropylene	0.91	0.02–0.1	5–20	1–8	400	25
Polyethylene[d]	0.95	0.02–0.2	5–50	117	2588	10
Basalt[e]	2.6	0.011	0.005–continuous	7.6	1000	2.6
Cellulose[a]	1.5[a]	0.02–0.12	0.5–5	10–50	300–1000	20

[a] Mehta and Monteiro (2004) and Hannant (2003).

[b] Bank (2006).

[c] Lee (1983).

[d] Lu et al. (1998).

[e] Sim et al. (2005).

or even higher (Otter and Naaman, 1988). Swamy and Mangat (1975) concluded that the most significant role of fibre reinforcement lies in increasing the postcracking properties of ductility, tensile strain capability and energy absorption capacity. Only marginal increase in compressive strength of high-strength concrete was achieved when steel fibres at 1% by volume were added (Nataraja et al., 1999). Much more significant improvements have been recorded on indirect tensile strength, modulus of rupture and toughness values (Nataraja et al., 1999).

The influence of certain geometric characteristics of fibres (Naaman and Najm, 1991; Banthia and Gupta, 2006; Banthia et al., 1994a,b; Szwabowski and Ponikiewski, 2008) and their alignment in the matrix have been recognized to affect the performance of fibre reinforcement (Banthia and Trottier, 1994). Prominent among these characteristics are the diameter, the length, the ratio of length to diameter (known as the aspect ratio l/d), the shape of the cross section and the presence or absence of hooks at the ends of the fibres. These characteristics are believed to significantly influence the bond strength between the matrix and the fibres and thus influence all the mechanical properties of the composite.

The influence of geometry and cracking have been studied and reported since the 1970s (Swamy and Mangat, 1974, 1975; Swamy et al., 1974). Steel fibres in particular and metallic fibres in general present an easier task for geometrical engineering in order to maximise the performance with the cementitious matrix.

The effect of the aspect ratio on compressive strength has been extensively studied. Fig. 21.1 shows such influence when steel-fibre volume was 2% of the concrete mixture (Otter and Naaman, 1988). It can be seen that the larger the aspect ratio, the higher the compressive strength. However, a more significant effect of this criterion has been observed on toughness of the composite material. The area under the stress–strain diagram significantly increases with the increase of the aspect ratio. This property results in reducing the brittle nature of failure.

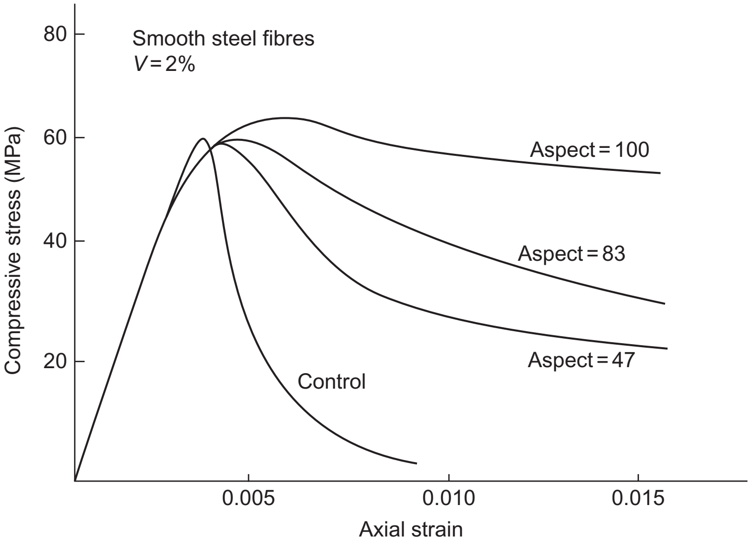

Fig. 21.1 Effect of aspect ratio on the compressive strength of concrete (Otter and Naaman, 1988).

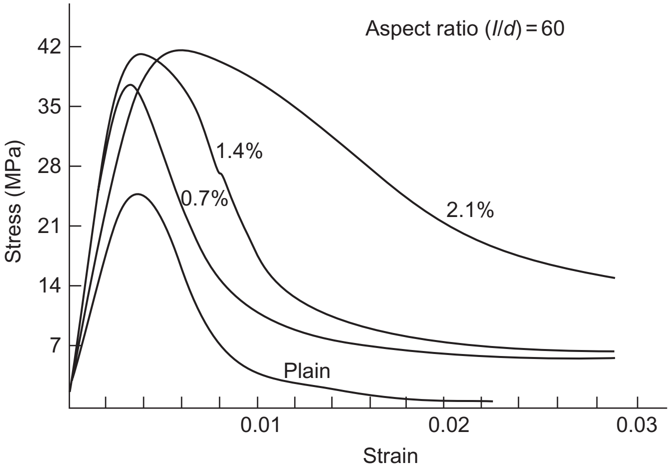

Fig. 21.2 Effect of increasing the proportion of steel fibres on compressive strength. Courtesy of Otter, D.E., Naaman, A.E., 1988. Fiber reinforced concrete under cyclic and dynamic compression load. Research report no. UMCE 88-9, Department of Civil Engineering, University of Michigan, Ann Arbor. University of Michiga, Michigan-Anne Arbor, 178 pp.

Many researchers have reported the effect of the quantity of fibres in relation to the volume of the composite (Ganeshalingam et al., 1981; Zollo, 1997). Fig. 21.2 summarises this effect with steel-fibre additions (Otter and Naaman, 1988). It shows that there is no linear relationship between the volume proportion of fibres and the gain in compressive strength. It is observed that compressive strength increases quite substantially with the addition of only as little as 0.7% by volume of steel fibres. However, there is no linear relationship between the gain in compressive strength and the quantity of fibres. The difficulty that fibre inclusion causes during the initial mixing, including balling of fibres and compacting problems, results in inadequately compacted and air-entrapped concrete. Nevertheless, the increase in fibre quantity results in a significant increase in strain energy density and thus imparts increased ductility to the composite.

Perhaps the behaviour of fibre-reinforced concrete in flexure and under tensile stresses is more important as far as structural engineers are concerned. Thus, a large volume of research has been done to study these issues. The factors that influence compressive strength also affect flexural and tensile behaviour, albeit to different extents. Most important of these factors are: the diameter d, the length l, the volume fraction V, the aspect ratio l/d, the surface geometry, and, of course, the fibre material and its properties.

Several models have been postulated in order to predict the ultimate strength in flexure (Hannant, 1978). A simple and quite accurate model has been suggested by Shah and Rangan (1971) as follows:

$$f_{\mathrm{cc}} = Af_{\mathrm{m}}\left(1-V_{\mathrm{f}}\right)+B\left(V_{\mathrm{f}}L/d_{\mathrm{f}}\right)$$

where f_{cc} is the ultimate tensile strength of the fibre composite, f_m is the maximum strength of the plain matrix (mortar or concrete), A and B are constants that can be determined experimentally. For plain concrete, $A=1$ and $B=0$. The constant B accounts for the bond strength of the fibres and randomness of fibre distribution. V_f is the volume fraction of the fibre, L is the fibre length and d_f is the fibre diameter.

Swamy et al. (1974) determined values for A and B at two stages of loading in flexure. The first is at the formation of first crack, being the demarcation of linear behaviour. The values of A and B at this stage are 0.843 and 4.25, respectively. The second stage is that of ultimate flexural capacity. The values of A and B at this stage are 0.97 and 4.94, respectively (Swamy et al., 1974; Zia et al., 1994).

Generally, the tensile strength is significantly improved by the inclusion of fibres. Short carbon, aramid and vinylon fibres were admixed at volume percentages ranging from 1.86 to 3.40 and caused increased tensile strength with the increase in fibre content (Nakagawa et al., 1989). Tests conducted on carbon short fibres admixed with mortar at just 0.5% by volume resulted in a tensile strength increase of more than 100% compared to plain mortars (Zheng and Chung, 1989).

21.4.2 Cracking in ordinary reinforced concrete and the advantage of fibres

It is known that conventional concrete has a very low tensile strength, while it possesses quite a high strength in compression. The reinforcement of concrete was introduced to compensate for its low tensile strength, which was considered unavoidable. In reinforced concrete, it is usually the concrete that carries the compression, while the tensile stresses are nearly all taken by the reinforcement. In a typical case of a downward-loaded reinforced concrete beam, for example, the portion above the neutral axis of the beam is under compression, while the part below the neutral axis is under tension and is assumed to be cracked. The force is taken by the reinforcing bars only, while the function of the surrounding concrete is to protect the steel rods from corrosion and to bond and hold them in position so that they act as an integral part of the beam.

The reason behind the low tensile strength of the concrete is the presence of flaws in the body of the concrete. Under tensile stresses, these flaws tend to propagate and enlarge to form cracks, which transfer no tensile forces. Those cracks tend to enlarge and travel to the neutral axis. Their depth depends mainly on the magnitude of stresses, the depth of the beam and the quantity of reinforcement. The more the beam is loaded, the greater are the stresses. On the other hand, the deeper the beam, the greater the moment of inertia and hence the lower are the stresses. Also, if the quantity of reinforcement is increased, the beam can take greater stresses with less deflection and cracking. The function of the reinforcing rods, therefore, can be seen as to stitch up the cracked concrete parts. It could be concluded, therefore, that the disadvantageous property of low tensile strength of concrete causes (1) an increase in dimensions to protect the steel rods and (2) an increase in dimensions to ensure safe stresses that will not cause deep cracks and reduce durability. (3) Consequently, nearly half of the reinforced concrete flexural member is not carrying any load, but only adds to the dead weight that must be carried.

Efforts were therefore directed at improving the tensile strength of concrete itself. The idea of fibre inclusion supplied a good field for investigation. Theoretical considerations were given to explain the behaviour of concrete when reinforced with fibres. Two separate mechanisms have been proposed to explain the increase in tensile strength that can be obtained by fibre reinforcement. The first is the mechanism of a two-phase material, and the second is the classical reinforcement mechanism similar to usual reinforced concrete. These two mechanisms will be briefly described here.

21.4.2.1 The concept of a two-phase material and the crack-arrest principle

The conventional reinforced concrete is not a true two-phase material because the existence of one material (steel) does not improve the properties of the other material (concrete). It only takes the duty that the concrete is unable to take. If the properties of concrete itself could be improved by the presence of another material, a two-phase material could then be said to be attained.

Since the propagation of microcracks is the cause of low tensile strength, it is the prevention of this propagation that the researchers are after. This prevention was sought by applying the reinforcement in a certain way as to arrest the cracks in the flaws, allowing the concrete to exhibit its true tensile strength and, hence, creating a two-phase material. The basis of the crack extension force concept was advanced by Irwin et al. (Irwin et al., 1950; Irwin, 1970) as a measure of the influences that tend to extend a crack in a stressed medium. When a flaw propagates, it displaces material at its edges. If near these edges there is an obstacle rigid enough, it will obstruct the displacement of the material and stop the cracking.

Consider, for example, the case of applying tensile force on a plate using rigid grips (Romualdi and Batson, 1963). It is assumed that the grips uniformly transfer load to the plate ends. If a narrow slot is made perpendicular to the direction of the force, the crack tends to extend. The extension of crack tips is accompanied by irrecoverable energy losses due to the extension of the plastic zone in the vicinity of the crack tips. The rate of loss of irrecoverable work with respect to the crack extension is $\delta W/\delta A$, where W is the work needed to extend the crack, and A is the crack area. This quantity is a property of the material and may be described as the 'fracture toughness'. As the crack extends under fixed grip conditions, there is a reduction or release of stored elastic energy. The rate of release of elastic energy is denoted in the literature by the symbol G. For the particular system of this example, and based on the theorem and work of Griffith (1921),

$$G = \left(\pi\sigma^2 a\right) / E$$

where σ is the overall plate stress, a is half the crack length and E is the modulus of elasticity (Irwin, 1970).

The criterion for rapid crack propagation is that the rate of release of elastic energy becomes equal to or exceeds the rate of consumption of energy of the extending crack; that is,

$$G \geq \delta W / \delta A$$

If, however, a pair of stiffeners is riveted to the plate in the direction of the applied tension and perpendicular to the crack tips, the plate tends to stretch more than the stiffeners, due to the stress concentration at the crack tips. The rivets by virtue of their shear and bearing strength capacities resist this tendency (Romualdi and Batson, 1963).

The parallel to this arrest crack mechanism can be assumed in the case of fibre-reinforced concrete. Consider a mass of concrete subjected to a tensile stress σ, and reinforced with continuous wires running parallel to the applied stress. A crack of length $2a$ located between two wires would resemble the situation described above, where the wires take the role of the stiffeners. The forces that oppose the cracking were calculated and it was found that the stress required to extend a crack beyond the area enclosed by adjacent reinforcing rods is inversely proportional to the square root of the rod spacing. It was also found that when the spacing between the wires is less than 12 mm, the effect is significant and the tensile strength increases markedly (Romualdi and Mandel, 1964).

21.4.2.2 The concept of mixtures properties

It can be stated generally that the cement composite (ie, cement-based materials containing fibres) strength is increased by:

1. an increase in the modular ratio (E-fibre/E-matrix);
2. an increase in the fibre content;
3. an increase in the aspect ratio (length of fibre/diameter of the fibre) and
4. the degree of fibre alignment with the stress direction.

According to this concept, the combination of cement paste and steel fibres should produce a satisfactory composite with volume fractions in excess of 5%, while cement paste and glass fibres should produce a satisfactory composite only with volume fractions in excess of 12% (Al-Kayyali, 1973).

Thus, it is necessary that the conditions of either of these two approaches be fulfilled for the purpose of increasing the tensile strength of the concrete composite. This would necessitate adding a percentage of fibres that results in spacing less than 10 mm or in volume fractions greater than critical. The percentage of fibres needed is more than can be added practically due to mixing problems that arise while trying to disperse the fibres uniformly through the mix and would cause great compaction difficulties. Therefore, it can be expected that if ordinary techniques for mixing and compaction were used, the percentage of fibres that can be added would fall short of the percentage needed to increase the tensile strength. However, recent developments in concrete technology have renewed the interest in the ability to produce composite mixtures that overcome the aforementioned difficulties. These developments are discussed later in this chapter.

21.5 Applications of fibre-reinforced concrete

Fibres in general have been extensively and successfully used in floors more than in columns. About 60% of the approximately 100 million cubic metres of fibre-reinforced concrete annually produced are slabs on grade (Mindess, 2008). Shotcrete constitutes 25% of this production while precast and other applications take up the remaining 15% (Mindess, 2008). The relative success of this type of concrete is due to the ability to impart ductility, resist sudden collapse and earthquake loadings and preserve durability and integrity in resisting adverse physical and chemical effects (Richardson, 2005).

Steel fibres have found large use in industrial floors where they could completely replace the traditional reinforcement mesh. Their benefits in this type of construction are the bearing strength, resistance to impact loads, resistance to cracking and abrasion and impermeability of the fibre-reinforced concrete. The inclusion of steel fibres in industrial floors and in pavements and construction that requires joint formation has proved very beneficial. The presence of steel fibres prevents spalling of joint edges in preformed and sawn joints (Hannant, 2003). Using steel fibres in shotcreting has proved to be very popular in tunnelling, repairing and pool construction. Fibres, in general, and steel fibres, in particular, are much in use in tunnels and in deep underground structures. especially in mines where strong and crack-free concrete is sought (Wang et al., 2008; Cominoli et al., 2006).

21.6 Performance of fibre reinforcement in concrete

Significant improvement has been achieved in several aspects of performance of plain and conventionally reinforced concrete when fibre reinforcement is added. Such improvement includes significant increase in tensile strength, fracture toughness, flexural strength and compressive strength. It also includes better resistance to crack propagation, better shock absorption, better service in earthquake zones, and better fire resistance. All of these desirable qualities and more, have been determined and reported by most of the researchers in the field and those who have been referenced here. Despite all these proven benefits, fibre reinforcement has yet to fulfil the aspirations that were behind its modern revival. This is the creation of concrete that possesses tensile strength, which would be substantial if not equal to its compressive strength. The creation of such a composite material would have, no doubt, revolutionized the building industry. This development has still to happen. The difficulties that arise on the path of this achievement are summarized in certain major aspects. These are: the practical difficulty of mixing enough fibres in the usual concrete mixture, the necessity of obtaining high bond strength between the fibres and the concrete matrix, the difficulty of manufacturing corrugated fibres, especially if they were nonmetallic, the balling up of fibres in general and steel fibres in particular, the air entrapment and the cost effectiveness.

Nevertheless, there are factors that are influencing another strong come-back of fibre-reinforced concrete. High-strength concrete has become the norm in modern day construction. However, high-strength concrete is even more brittle than lower ‘normal’ strength concrete (Price, 2003). Other problems with high-strength concrete include autogenous shrinkage (Aitcin, 1999; de Larrard and Le Roy, 1992) and spalling in fire incidences (Kodur, 1999; Ongah et al., 2003). These issues and others will be presented in the following sections. They constitute in this author’s opinion, enough justification for a serious revisit of this important type of composite materials.

21.6.1 A new era in the field of fibre-reinforced concrete

With the fast development in high-performance concrete (HPC), the usage of fibres gained special attention. Naaman and Reinhardt (2003) have very well defined research in this area and categorized cementitious composites into four categories. These are deflection softening, deflection hardening, tensile hardening and high-energy absorption (Naaman and Reinhardt, 2003). Further elaboration on these categories has been published very recently (Wille et al., 2014) in which the authors illustrated the aforementioned categories. For this purpose, they used the following nomenclature:

σ_{cc}: cracking stress,
ε_{cc}: cracking strain,
E_{cc}: elastic modulus,
σ_{pc}: postcracking strength,
σ_{f1}: modulus of rupture,
σ_{f2}: equivalent bending strength and
g: energy absorption capacity.

They demonstrated the concept as shown in Fig. 21.3, which represents the results of their research. Although the figure is selfexplanatory, it deserves a few comments for further explanation.

> Level 0 in the figure shows the reference situation where no fibres are added. The failure is sudden and characterized by sudden strain and deflection softening with catastrophic collapse.
>
> Level 1 represents the situation where fibres are added but the behaviour exhibited strain softening as well as deflection softening. Although the failure might not be catastrophic, it is characterized by mainly the widening of a single crack in both cases of direct tensile and flexure.
>
> In Level 2, one can notice that failure in direct uniaxial tension is still characterized by the formation and widening of a single crack and thus by strain softening. However, flexural failure is characterized by the formation of several cracks and the start of the phenomenon of deflection hardening whereby the load had to increase for the deflection to also increase. Thus, the ultimate bending strength is higher than the modulus of rupture.
>
> Level 3 represents the case where the introduction of fibres resulted in creating a strain-hardening situation (albeit small) in the case of uniaxial tension, and a significant deflection hardening in the case of flexure. The failure in both cases is manifested with multiple cracking.
>
> Level 4, on the other hand, represents the situation where significant strain-hardening occurs in the case of uniaxial tension as well as in the case of flexure. The multiple cracking phenomenon is expected, in these cases, to be much more prolific. The category of composites that exhibits this level of behaviour is the one sought after when designing a cementitious composite.

So as to make clear distinction between categories of cementitious composites that may fulfill strain-hardening and deflection criteria, Wille et al. (2014) have suggested the value of g to be the defining value that may guide such distinction. In a tentative suggestion, they propose the value of $g \geq 50\,\mathrm{kJ/m^3}$ (Wille and Naaman, 2010) to be the value that places the composite member into the highest performance level.

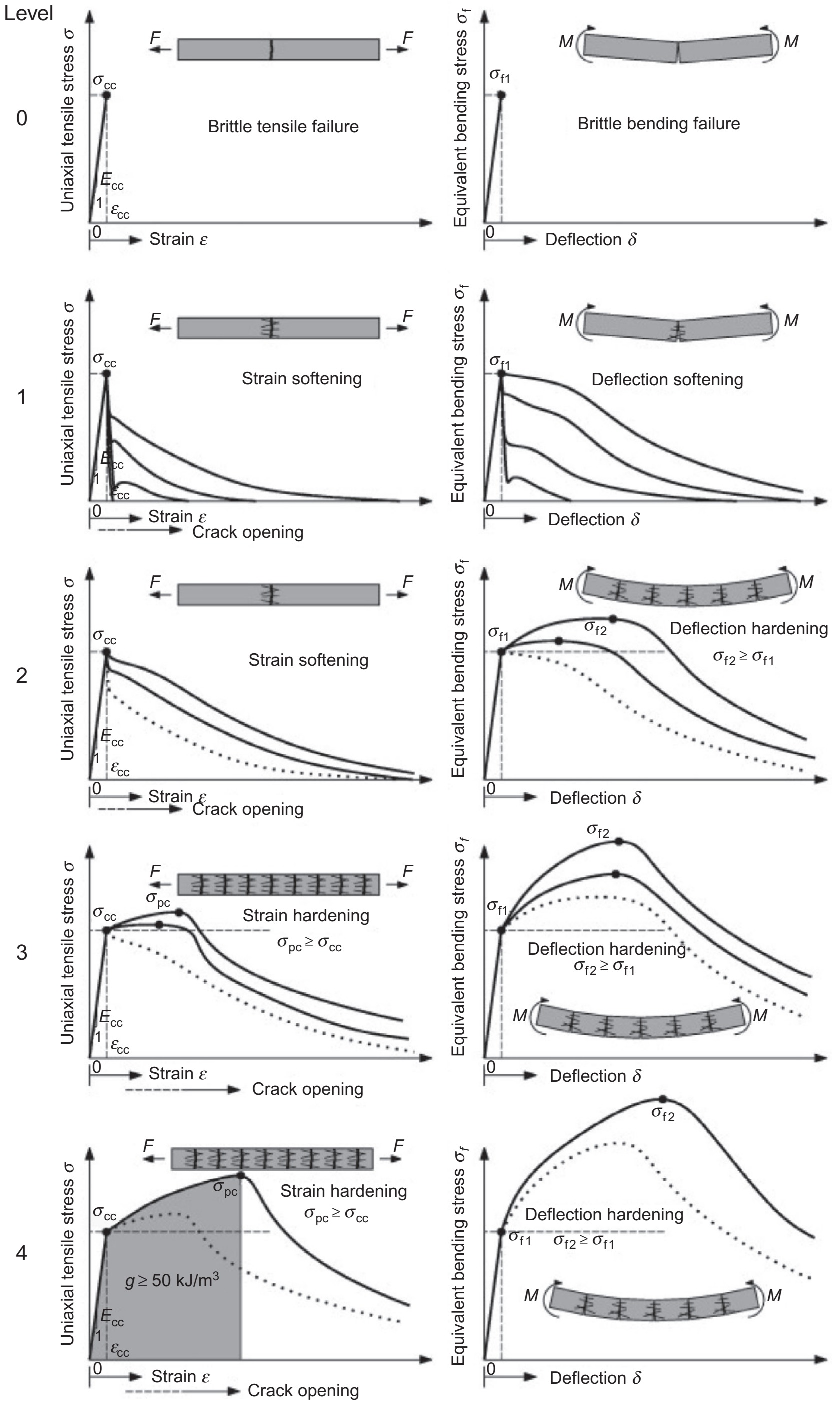

Fig. 21.3 The four categories of stress–strain and cracking in fibre-reinforced concrete composites (Wille et al., 2014).

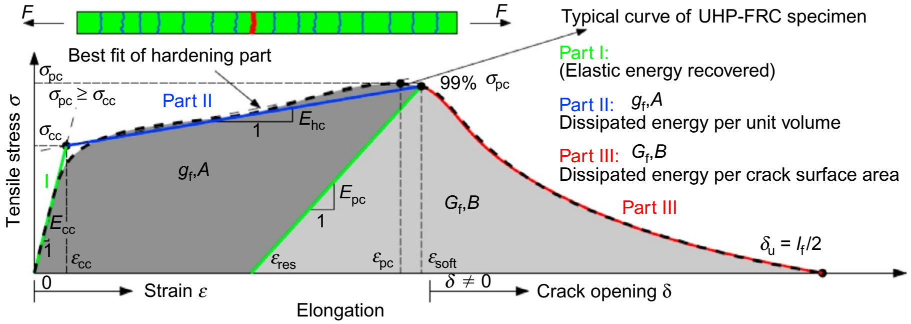

Fig. 21.4 A model for the strain-hardening category for direct tension in high-performance, fibre-reinforced concrete (Wille and Naaman, 2010; Wille et al., 2014).

A model for the strain-hardening category has been suggested (Wille and Naaman, 2010; Wille et al., 2014) and is illustrated in the diagram shown in Fig. 21.4. This figure shows the model of a strain-hardening, fibre-reinforced concrete. This is characterized by a relatively short strain period in the totally elastic regime followed by a smooth transition into a long period of strain-hardening that is, in turn, followed by a comparatively long period of strain softening. The most important visual feature of the behaviour is the uniform distribution of multiple hairline cracks accompanied by considerable elongation and followed by crack widening before failure in a typical ductile failure mode.

21.7 Recent developments

The past decade has seen several important developments that can be considered groundbreaking in the field of concrete manufacture. These developments have been instigated mainly by the acute need for reducing carbon dioxide emission and for producing sustainable concrete. The major advances in this field are briefly described in the following sections.

21.7.1 The development of selfcompacting concrete and its effect on fibre reinforcement

The advances achieved in developing selfcompacting concrete have been realized as a result of spectacular successes in manufacturing very effective chemical admixtures, namely, superplasticisers and viscosity-modifying agents (Okamura and Ouchi, 1999). As the major obstacle in including a large quantity of fibres has been the difficulty of working fresh mixes, it is evident that this development is of utmost importance in the field of fibre-reinforced concrete (Ponikiewskia and Katzerb, 2014). Ponkiewski and Katzerb produced selfcompacting fibre-reinforced mixes using a maximum of 0.9% by volume of polymer fibres or 1.5% steel fibres. Lin et al. (2014) experimented with seven types of fibres, metallic and nonmetallic, with different configurations. They were able to easily produce selfcompacting mixes with 2% volume content (Lin et al., 2014).

It must be noted, however, that their testing was confined to pastes while Ponkiewskia and Katzerb used concrete mixes with traditionally used aggregates. Lin has also produced high-performing, selfcompacted, fibre-reinforced concrete with a very high proportion of fly ash replacing portland cement (Lin, 2013). This is considered a very desirable type of construction as far as sustainability is concerned.

Consequently, and in appreciation of the importance of this development in the future of fibre reinforcement, mix-design for selfcompacting, fibre-reinforced mortars (Pereira de Oliveira et al., 2013) and concretes have been proposed (Ferrara et al., 2007; Deeb et al., 2012, Soltanzadeh et al., 2015). This field of research is ripe for further substantive development.

21.7.2 The development of hybrid fibre reinforcement

The idea of using more than one type of fibre in the same mix has been around since the mid-1970s (Walton and Majumdar, 1975). It has been observed that certain fibres, especially those of organic origin, imparted favourable properties to concrete in resisting impact forces (Goldfein, 1963, 1965). This characteristic is due mainly to their low elasticity modulus and high extendibility before failure. On the other hand, such organic fibres did not impart significant improvement in strength (Walton and Majumdar, 1975). Walton and Majumdar have, therefore, suggested the combination of more than one type of fibres so as one type would compensate for properties that are lacking in another type.

More recently, research has been published with essentially the same aim of creating synergy using organic and metallic fibres (Banthia et al., 2014a). They have advocated the possible use of several types and geometries of fibres in the same mix provided that the design is done rationally so as to optimize the synergy created by the hybridization.

21.7.3 The development of reactive powder concrete

Reactive powder concrete (RPC) was developed by the scientific team of BOUYGUES in France (Richard, 1996), and was able to achieve compressive strengths ranging between 200 and 800 MPa. Its flexural tensile strength ranged from 70 to 140 MPa. The ductility of this concrete as measured by the fracture energy value is 30,000 J/m^2 compared to ordinary concrete with just 120 J/m^2. A main ingredient of RPC is the inclusion of straight steel fibres of approximately 13 mm length and a diameter of about 0.15 mm. The quantity of fibres in the mix is between 1.5% and 3% by volume. Although this material has been capable of producing very elegant structures, its initial materials and manufacturing costs have prevented a potential widespread usage.

21.7.4 The development of geopolymer concrete

Geopolymer concrete has become a very popular subject of research in recent years. The main reason for its popularity is that the use of geopolymers carries the potential of totally replacing portland cement as the cementitious material. What makes it even more attractive to use is the fact that its raw component is usually an industrial by-product rich with silica and alumina. Such compounds are found in abundance in by-products like fly ash, slag, rice husk ash and metakaolin.

Ground granulated blast furnace slag used as the only alkali-activated material (Mozgawa and Deja, 2009; Li et al., 2010; Oh et al., 2010) or one of combinations of alkali-activated industrial by-products (Cheng and Chiu, 2003; Izquierdo et al., 2009) has produced expected promising results as load-bearing and fire-resistant material. Rice husk ash has been researched mainly in combination with fly ash to produce satisfactory performing geopolymers (Kusbiantoro et al., 2012; He et al., 2013). The use of red mud as a major geopolymer raw material has been explored (van Riessen et al., 2013) and is indeed a very promising waste material that can be utilized in combination with different by-products in sectors of construction that also result in environmental protection (He et al., 2012, 2013; Kumar and Kumar, 2013; Hajjaji et al., 2013; Badanoiu et al., 2015). Metakaolin has become an important supplementary cementitious material that possesses geopolymeric components. Metakaolin has been activated on its own to act as the only cementitious material (Rovnanik, 2010; Kuenzel et al., 2013; White et al., 2013; Pelisser et al., 2013; Li et al., 2010) or in combination with fly ash (Khatib et al., 2014; Papa et al., 2014) or with rice husk ash and silica fume (Villaquiran-Caicedo et al., 2015).

Arguably, the most popular material used to produce geopolymer concrete is fly ash. This has prompted research into optimizing mix-design methods for this new construction material (Junaid et al., 2015). Nevertheless, significant research into the use of other silica- and alumina-bearing compounds has been increasing.

An expected corollary to these activities has been exploring the possibility of fibre reinforcement in geopolymer concrete (Nematollahi et al., 2014). The feasibility of using fibres in geopolymer concrete has been clearly established with fibre-reinforced geopolymers displaying a clear strain-hardening characteristic with strain values in the range of 4% (Ohno and Li, 2014). Fig. 21.5 from the work of Ohno and Li (2014) shows the behaviour of a fibre-reinforced geopolymer concrete bar in direct tensile test.

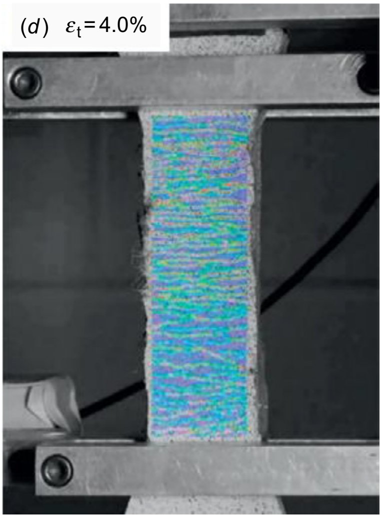

Fig. 21.5 Fibre-reinforced geopolymer concrete showing the multiple cracking phenomenon (cracks are marked by blue lines) (Ohno and Li, 2014).

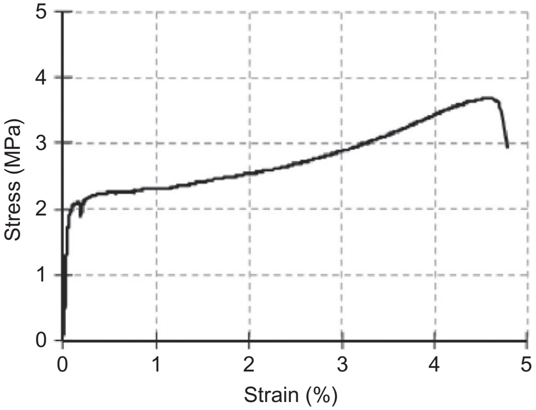

Fig. 21.6 The stress–strain diagram corresponding to fibre-reinforced geopolymer concrete showing clear strain-hardening characteristic (Ohno and Li, 2014).

Fig. 21.5 very clearly shows the phenomenon of multiple cracking that distinguishes the strain-hardening phenomenon (Wille and Naaman, 2010). Ohno and Li (2014) further demonstrated this phenomenon in the diagram of stress–strain shown in Fig. 21.6. The strain-hardening phenomenon is clearly demonstrated in which the material has acquired approximately 4.7% of strain while stress has increased constantly to nearly double its value at the point of maximum elastic stress. This diagram confirms the results of Wille and Naaman and their proposed model (Wille and Naaman, 2010).

21.8 Concrete sustainability and the role of fibre reinforcement

The average global concrete usage is 1 m^3 per person (Aïtcin, 2000). The average cement consumption of each person per year has been estimated as 268 kg (Kayali et al., 2008). Estimates of the amount of CO_2 that results from the manufacture of cement vary from 0.8 to 1.0 tonne of CO_2 per tonne of cement produced (Gartner, 2004). Recent studies estimate that the cement industry contributes 5–8% of the total CO_2 emissions (Huntzinger and Eatmon, 2009; Scrivener and Kirkpatrick, 2008). Any reduction in this undesirable contribution would be welcome.

There are many ways by which such reduction may be achieved. One very important strategy is to construct buildings with durable concrete that does not need costly maintenance or even demolition and reconstruction in a relatively short period. Another strategy is to be creative in realizing the potentials of new materials and techniques that are utilized in the industry and to dedicate an important portion of research into such new materials.

It is easy to argue that protecting construction from deterioration is a major key to avoid repair or reconstruction that, in turn, needs additional cement. Thus, durability of concrete is necessary for sustainability. Deterioration of reinforced concrete structures is caused by many physical and chemical factors. Among the physical factors are fire exposure, freezing and thawing, abrasion, and thermal and moisture influenced deformation and cracking. Among the chemical factors are the alkali silicates reactivity (ASR), sulphate attack and corrosion of reinforcement. The benefits on durability of including industrial by-products like fly ash (Malhotra, 1990), rice husk ash (Antiohos et al., 2014), silica fume (Alexander and Magee, 1999), metakaolin (Al-Akhras, 2006; Khatib and Clay, 2004; Khatib and Wild, 1998) and blast furnace slag (Kayali et al., 2012; Ahmed et al., 2009; Bernal et al., 2012) in concrete have been explored for many years. The advantages derived from such inclusion render these products to be considered as necessary ingredients of concrete rather than just a way of reducing cost with fillers. There is ample evidence that ground granulated blast furnace slag, for example, can advantageously replace portland cement even in a proportion exceeding 70% while imparting desirable durability qualities to concrete structures (Hinczak, 1991; Neville, 1995; Kawamura and Takemoto, 1988; Ingerslev, 1989). Malhotra has been a prominent pioneer of using high-volume fly ash to replace portland cement (Malhotra, 1999). Silica fume has assumed a very prominent role in the production of high- and ultrahigh-strength concrete to the extent that it has become much more expensive than portland cement. Kayali et al. (2012) have shown that using ground granulated blast furnace slag results in producing hydrotalcite as a main hydration product. They have found that hydrotalcite plays a vital role in preventing chloride-initiated corrosion of steel reinforcement (Kayali et al., 2012). The prospect of including these alternative and supplementary materials in concrete need to be extensively studied in relation to the developments in fibre reinforcement and the innovation that is taking place in concrete composites.

Kayali (2004) has explored the effect of high-volume fly ash on the role of steel fibres and found that the use of high-volume fly ash in fibre-reinforced concrete has resulted in large improvement in its mechanical characteristics. This is attributed to the pozzolanic activity of the fly ash that replaces the preferentially oriented crystalline layer of calcium hydroxide in the interface, with denser hydration products that impart greater strength to the matrix and higher bond capacity between the matrix and the fibres. Kayali concluded that the beneficial effects of fibre reinforcement can be achieved when the concrete contained a large amount of fly ash (Kayali, 2004). Lin (2013) has recently demonstrated that high-performance, high-volume fly ash binder could manifest strain-hardening characteristics with fibre reinforcement (Lin, 2013). Cui and Kayali (2013), being aware of the imminent introduction of geopolymers to the concrete industry, examined the bond strength of the geopolymer matrix with the reinforcement, whether it is reinforced traditionally with steel bars, reinforced solely with fibres, or reinforced with both fibres and traditional steel bars (Cui and Kayali, 2013). They reported the best bond to occur between the concrete and the reinforcing bars was when the matrix is made of fibre-reinforced geopolymer.

Naaman and Najm (1991), Naaman and Reinhardt (1996), Wille and Naaman (2010) and Wille et al. (2014) have pioneered the development of high-performance, strain-hardening, fibre-reinforced concrete (Naaman and Najm, 1991; Naaman and Reinhardt, 1996; Wille and Naaman, 2010; Wille et al., 2014). Li (1993, 2003) and Li and Kanda (1998) have extensively investigated the potentials of this new generation of fibre-reinforced composites. They demonstrated that fibre-reinforced composites containing 2% fibres by volume, which have a ductility of 3–4% in direct tension can be easily manufactured. They gave this type of composite the name 'engineered cementitious composites' (ECC) (Li, 1993, 2003; Li and Kanda, 1998). The material is engineered on the basis of micromechanics, which permits optimizing matrix strength and fibre–matrix bond strength (Mindess, 2008). It efficiently resists cracking and significantly improves durability. Moreover, ECC can be cast using pumping as well as using shotcreting and is also able to be placed as selfcompacting concrete. Lin (2013) and Lin et al. (2014) have succeeded in producing high-volume fly ash strain-hardening, fibre-reinforced composite (Lin, 2013; Lin et al., 2014). This development carries a special importance for sustainability as it deals directly with the issue of utilizing waste material in the achievement of strain-hardening cementitious composites. It is furthermore promising to usher in another breakthrough in this important subject in efforts to provide sustainability in concrete construction.

The key issue that enabled the development of ECC and RPC has been the realization that the desired properties cannot be obtained without studying the micromechanics of the interface between the fibres, the aggregates and the matrix (Li, 2003). Nor could such development be obtained without the microstructural analysis of the materials, their development with time, and their interactions with other ingredients down to the micro- and nanometric scales (Dugat et al., 1996). In his review of ECC, Li (2003) demonstrates the importance of engineering the fibres in order to arrive at the correct material property that ensures a ductile behaviour (Li, 2003). Based on the concept of integrated structures-materials design (Li and Fischer, 2002), it has been possible to manufacture fibre-reinforced concrete of 41 MPa that structurally performed far better than conventional concrete of 50 MPa (Kesner and Billington, 2002).

The aforementioned developments in the science of fibre-reinforced concrete are expected to have far-reaching effects on the durability of structural concrete. Already, high-performance, fibre-reinforced cementitious composites have been successfully applied to the retrofitting of damaged concrete beams (Farhat et al., 2007). External strengthening of various elements of civil engineering construction has become very popular with the use of FRP composites (Pendhari et al., 2008). Fibre-reinforced plastics have become very much researched and successfully employed in retrofitting structures that were not initially designed to withstand seismic loads (Caterino et al., 2008). Moreover, the recent advances in the manufacturing of fibre-reinforced cementitious materials have made it possible to effectively resist seismic loading resulting in the saving of lives as well as structures (Fischer and Li, 2003).

The significant reduction in crack width and permeability that the engineered fibre-reinforced concrete can achieve has made it possible to protect coastal structures against reinforcement corrosion (Lawler et al., 2002) and thus significantly prolong the life expectancy of structures.

However, an important aspect that needs to be addressed is that of cost versus long-term benefits. It is often the case that the initial costs of durability-effective materials and construction significantly exceed the cost of structures where the issues of structural ductility and durability have not been seriously addressed and considered in design. The need for constant inspection, reparation and eventual replacement of such structures carries costs that may far exceed the initial value. It is therefore evident and timely that engineers, contractors, scientists, educators, law makers and regulators need to address the issue of sustainability and include it as an important factor in estimating the cost of structures that should include both the initial and service life cost estimations.

21.9 Future trends

From the previous sections of this chapter, it may be encouraging to conclude that concrete researchers have been fully aware for many years of the important issues that threaten concrete sustainability. Research into these issues has led to fundamental progress, which in turn is expected to serve the economy and science of cementitious materials as well as sustainability. It is also sobering that a method of building that can easily be described as 'primitive' has come-back to life, thanks to the drive toward achieving more sustainable construction. The idea of including straw and other organic fibres that humans used in ancient times has now reemerged albeit with far more sophistication. Research into using nanofibres and nanotubes in concrete construction has recently taken a significant leap (Sbia et al., 2015; Konsta-Gdoutos, 2013; Peyvandi et al., 2013; Puentes et al., 2014). The role of carbon nanofibres in producing alternative concretes was assessed in relation to early age shrinkage and cracking. It was found that combining such concrete with micropolypropylene fibres eliminates shrinkage cracking risk (Puentes et al., 2014). Sbia et al. (2015) found that including carbon nanofibres in concrete mixes results in increasing compaction density and strength (Sbia et al., 2015). Konsta-Gdoutos (2013) demonstrated the advantages to concrete strength, of including carbon nanofibres as well as carbon nanotubes (Konsta-Gdoutos, 2013). She has also demonstrated the benefits of adding silica nanoparticles so as to solve the calcium leaching problem (Konsta-Gdoutos, 2013).

It is very well known that many factors can deleteriously affect concrete structures. The economical, environmental and social factors that determine the sustainability of structures or their materials often necessitate repairing existent construction. Concrete repair must itself be durable. It must also be thoroughly based on deep understanding of the problems that cause deterioration. The materials used in repair must provide solutions that simultaneously satisfy architectural and safety demands. The repair job must correctly blend and bond with the original material. It must protect the original substrata and prevent the deterioration processes from reinitiating. These requirements have made fibre-reinforced concrete an obvious choice for repair tasks, and thus the extensive research needed in this area. The usage of ultrahigh-performance, fibre-reinforced concrete presents a first choice material for such purpose (Tayeh et al., 2013). Banthia et al. (2014a,b) have highlighted the approach by which a repair material should be examined. They emphasized that the repair material should possess resistance to the cause of deterioration that required the repair, bonding to the original material, durability and compatibility with the substrata. They further reported evident success of fibre-reinforced cementitious composites in providing the means to achieve these objectives (Banthia et al., 2014b). The importance of the bond between fibre-reinforced matrix and conventional reinforcing bars has been reported (Cui and Kayali, 2013) and further examined in the particularly relevant context of structural concrete repair (Zanotti et al., 2014).

The quest for reducing weight and volume of concrete structures has become more urgent. This is driven by several factors that directly or indirectly impact on the sustainability of concrete structures. One of these is the need to reduce cement consumption. Another factor is the endeavour to maximize rentable spaces. A third factor is the attempt to reduce foundations work. A fourth factor is to minimize the cost of construction, and a fifth is to produce slender and architecturally elegant structures. All of these issues require optimizing materials' mechanical characteristics and their structural and architectural functions. Again, it may be seen easily that ultrahigh-strength, fibre-reinforced cementitious materials possess the potential to satisfy the conditions mentioned previously. Ritchie and Kayyali (1975) reported their research into the effects of fibre reinforcement on lightweight aggregate concrete. At that time, the use of superplasticisers was not widespread and thus the workability difficulties that the fibres created in concrete mixes limited the use of such composite (Ritchie and Kayyali, 1975; Swamy and Jojagha, 1982b). Later on, the research into this type of composite acquired further interest especially in the role of fibres in reducing cracking in early and mature concrete (Kayali et al., 1999, 2003) and resisting impact forces (Swamy and Jojagha, 1982a; Ritchie and Kayyali, 1975). Parallel advances in the creation of lightweight concrete in which mechanical properties are not compromised have also been achieved (Kayali, 2008). This led to a new revival in the research to combine the benefits of light weight and fibres (Bonakdar et al., 2013; Mara et al., 2014).

Using fibre-reinforced concrete to protect from spalling in fire incidents has started to gain interest, especially with pumping construction (Heo et al., 2011). This usage is also very much related to the use of fibre-reinforced normal weight and lightweight concrete in structures that must resist seismic forces. Ultrahigh-strength, strain-hardening, fibre-reinforced concrete is a very promising material for future buildings that are expected to resist seismic and explosive shocks. Research in this area is already under way at an accelerating pace and the following research references are only a small representative of the burst of activity in these fields (Abbas et al., 2014; Aoude et al., 2015; Huang et al., 2015; Roehm et al., 2015; Zhang et al., 2013).

It can therefore be easily concluded that the spectacular advances in chemical engineering made it possible for concrete mixtures to be designed to an optimum of particle distribution, waste material inclusion, minimization of cement content, and maximization of fibre reinforcement in order to achieve concretes that are ductile, environmentally friendly, durable, safe and economical. In one word, 'sustainable'.

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Sustainability of wastepaper in construction

22

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22.1 Introduction

Humans have not always relied on paper to record human activity. This is witnessed by the inscriptions in prehistoric caves where some of the sketches are made to very high detail. The artistry to show events such as rain, wars, festivals, as well as accurate depiction of both dead and living things—mountains, animals, wind, fire, sun—shows an early urge to maintain an accurate record of things and events. The use of stone must have had appeal due to its durability, but the lack of versatility, especially upon migration, must have caused quite a headache. Early humans had to try to revisit earlier writings and/or resort to making fresh recordings. With natural disasters, such as wars, fires and floods, and migration, the need for mobile facilities must have gradually developed. This must have given rise to the use of slates and stone slabs, which with time were also found to be cumbersome and gave way to lighter materials. Some of the early recorded lightweight materials are from Egypt, where papyrus and other reeds were used. This must have caused quite a sensation and the material must have been a major invention and innovation at the time. Other parts of the world caught up, as witnessed by the discovery of scrolls of paper-like materials in numerous places with well-written and accurate recordings of human activity and other happenings. Not many signs exist of any recycling tendencies during this time, although perhaps the use of slates must have been triggered by this necessity, borne out of the need to wipe clean and reuse the material.

22.2 Modern paper manufacture

Modern paper manufacture uses either virgin pulp from vegetable matter (mainly from trees) or a combination of virgin pulp and recycled pulp composed of cellulose fibres sourced from recycled paper. Countries with potential for larger forested regions have the highest capacity for paper production (North American (United States and Canada), northern European (Finland, Sweden and North-West Russia); East Asian countries (Russia, China, Japan and South Korea); Australasia; and Brazil). The world total production as of 2011 was about 400 million tonnes; and Table 22.1 shows the top leading countries in the manufacture of paper and paperboard at that time. The table shows China as the world's leading producer of paper, having overtaken the United States in 2009 (Japan Paper Association (JPA), 2011).

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Table 22.1 Main producer countries of paper and paperboard

Rank 2011	Country	Production in 2011 (1000 tonnes)	Share 2011 (%)
1	China	99,300	24.9
2	United States	75,083	18.8
3	Japan	26,627	6.7
4	Germany	22,698	5.7
5	Canada	12,112	3.0
6	South Korea	11,492	2.9
7	Finland	11,329	2.8
8	Sweden	11,298	2.8
9	Brazil	10,159	2.5
10	Indonesia	10,035	2.5

Annual Review of Global Pulp and Paper Statistics by Resource Information Systems Inc./Recherche d'Information sur l'Internet (RISI), published by Japan Paper Association (JPA), 2011. Annual Review of Global Pulp and Paper Statistics by RISI (Resource Information Systems Inc./Recherche d'Information sur l'Internet).

22.3 Wastepaper

The paper industry is a vast consumer of virgin raw materials and has met with significant criticism for its share in environmental degradation. Vast tracts of virgin forest have been cleared to provide raw materials to meet the high and insatiable demand for paper and plasterboard. The situation has been exacerbated by claims of low wages and insensitivity to the loss of large tracts of native forests such as those in Brazil, Congo and Indonesia. Perhaps for these reasons, there has been a growing impetus to recycle paper to mitigate the consumption of virgin pulp. It is during the paper recycling process that other wastes result, such as wastepaper sludge and the ash that results from the combustion of the sludge. The ash is the subject of this chapter, and research on the utilization of this waste will be discussed. However, first, the production process of this waste must be discussed and/or explained.

22.4 Paper recycling

In order to reduce the environmental impact of paper manufacture, increasing quantities of paper are being recycled (Frederick et al., 1996). The part of the paper that is not recovered during recycling is the deinking sludge. According to Péra and Amrouz (1998), by 1998, the European community produced 5.8 million tonnes/year of wastepaper sludge, and the United States, over 1 million tonnes. Until recently, the resultant wastepaper sludge ash (WSA) has ended in landfill sites. The latent energy of the organic component (mainly residual cellulose fibres) can be recovered by combustion of the sludge. In practice, the combustion temperature is in excess of 850°C, thereby reducing the volume of waste going to landfill sites to around 40% or 50% of the original dry solid sludge. A common practice is to combust wastepaper sludge in a fluidized bed and utilize the resultant energy to run and operate the plant. There has been much research in the last two decades on the identification of possible potential applications of commercial WSA as a cementitious material. A significant number of these have attempted to blend WSA with ground granulated blast furnace slag (GGBS), or use WSA as a partial replacement for portland cement (PC) (Kinuthia et al., 2001a,b; O'Farrell et al., 2001; Bai et al., 2002, 2003; Veerapan et al., 2003; Chaipanich et al., 2005; Mozaffari et al., 2009; Nidzam and Kinuthia, 2010, 2011a,b; Nidzam et al., 2011; Wong et al., 2015).

22.5 Production and properties of wastepaper sludge ash (WSA)

22.5.1 Production

In practice, the commercial process of combustion of wastepaper sludge in a fluidized bed is very different from soaking at a constant temperature under laboratory conditions. In commercial production, there are bound to be many variations in composition due to variations in paper that is being recycled, which in turn depends on people's recycling habits and prevailing interests in printed media, such as consumption of glossy magazines and general newsprint. The calcining temperature of the wastepaper sludge in a fluidized-bed combustor is in part determined by the particular energy recovery system adopted (if employed) and also by the requirement of a maximum allowable limit for dioxin emissions.

For the UK-based ash reported in this chapter, the sludge enters the combustion zone with around 40% moisture, resulting with flue gases generally consisting of 25% residual water vapour (steam). Air is blown in at the bottom and it is occasionally necessary to burn gas in the air to preheat it. In the combustion zone, the temperatures can vary between 850°C and 1200°C, being cooled toward the top. Generally, the mixture will stay in the combustion zone for 3–5 s before going through a cooling zone, which cools the gases to around 200°C in about 3–5 s. This is designed to produce extremely low levels of dioxins. Samples of the ash suggest a total ITEC (international toxic equivalents) values (ppt) ranging from 0.80 to 1.15 ng/kg, which is significantly below levels observed in ashes from other similar combustion processes (eg, pulvarized fuel ash). The process is very similar to the flash calcining of kaolinite reported by Salvador (1995), which involves the very rapid heating and cooling of kaolinite powder suspended in air. In Salvador's work, a maximum temperature of up to 1000°C was employed with dwell times of 2–12 s. The pozzolanic activity of the resultant metakaolin was high for calcining temperatures of up to at least 980°C at a dwell time of 2 s. In contrast, for kaolin that was soak calcined (for 5 h), the pozzolanic activity fell rapidly to zero above 900°C.

22.5.2 Particle-size distribution

Fig. 22.1A and B shows the particle-size distributions of typical samples of WSA (Kinuthia et al., 2001b; Mozaffari et al., 2006). All three samples in Fig. 22.1A suggest uniformly distributed particle size, within the range of about 2–100 μm. The vertical bold lines in Fig. 22.1A show the mean particle sizes for ground WSA and for a typical PC. Relative to the typical particle size in PC (average particle size of about 15 μm), WSA is much coarser.

It is possible to observe the coarseness both visually and also by touch, with the fine sand-size particles providing a gritty feel between the fingers. The coarseness is attributed to

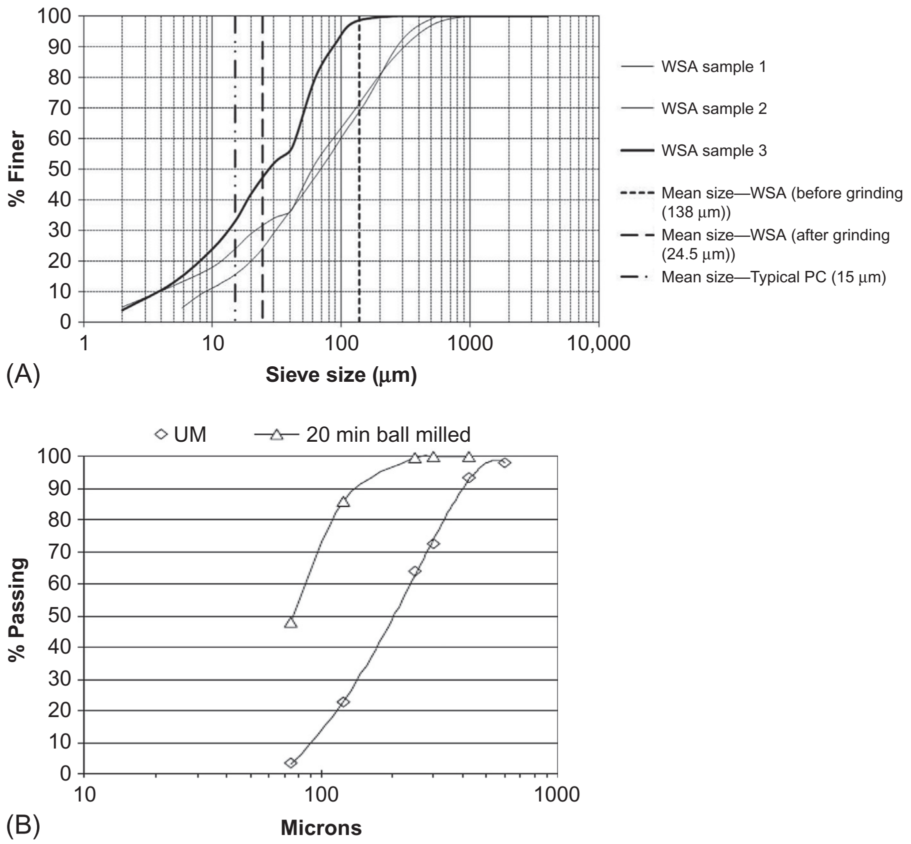

Fig. 22.1 (A and B) Particle-size distribution of WSA (vertical bold lines in (A) show mean particle sizes; UM: unmilled WSA).

From (A) After Kinuthia, J.M., O'Farrell, M., Sabir, B.B., Wild, S., 2001b. A preliminary study of the cementitious properties of wastepaper sludge ash (WSA)—ground granulated blast-furnace slag (GGBS) blends. In: Dhir, R.K., Limbachiya, M.C., Newlands, M.D. (Eds.), Proceedings of the International Symposium on Recovery and Recycling of Paper, Dundee University, 19th March 2001. Thomas Telford. ISBN: 0-7277-2993. (B) From Mozaffari, E., O'Farrell, M., Kinuthia, J.M., Wild, S., 2006. Improving strength development of wastepaper sludge ash by wet-milling. Cem. Concr. Compos. 28 (2006), 144–152.

the much coarser bottom ash from the fluidized-bed system, which is commonly intermixed with the fine ash that is extracted from the flue gases. The particle size of WSA varies, therefore, depending on the proportion of sand-size particles from the bottom ash. Kinuthia et al. (2001b) observed a mean particle size of 138 μm. They also attempted to grind the WSA, managing to reduce the fineness closer to that of a typical PC (mean particle size 15 μm) by reducing the average particle size to 24.5 μm. Mozaffari et al. (2006) attempted wet milling of WSA, achieving the particle size refinement shown in Fig. 22.1B. Attempting to grind a waste material is not a popular or recommended trend as it is normally an unattractive option economically, and attempts should made to utilize waste with minimal further processing.

22.5.3 Chemical composition

The dry wastepaper sludge is composed approximately of equal amounts of organic and inorganic components, the latter consisting principally of kaolin and limestone. The kaolinite content of wastepaper sludge can vary within the range of 15–75% and the calcite content from 21% to 70% (Péra and Amrouz, 1998). The composition is a function of the type, grade and quality of paper recycled, and the phase composition will be dependent on its thermal history. According to Péra and Amrouz (1998), kaolinite undergoes dehydroxylation above 600°C to produce metakaolin ($Al_2Si_2O_7$), which is a semiamorphous highly active pozzolan. At 925°C, it converts to crystalline spinel ($2Al_2O_3{\cdot}3SiO_2$) and free silica (SiO_2), and the crystalline end products at 1400°C are mullite ($3Al_2O_3{\cdot}SiO_2$) and cristobalite (SiO_2). Calcite ($CaCO_3$) decomposes above 700°C to give quicklime (CaO) and carbon dioxide (CO_2). However, heating calcite and kaolinite together will not necessarily result in a mixture of the above phases because quicklime and metakaolin react (depending on the temperature and soaking time) to form a wide range of calcium aluminosilicate phases. The two principal phases are gehlenite ($Ca_2Al_2SiO_7$) and anorthite ($CaAl_2Si_2O_8$), although at high calcite to kaolin ratios calcium silicate (larnite—Ca_2SiO_4) replaces anorthite. Larnite has latent hydraulic properties whereas gehlenite and anorthite are considered to be nonhydraulic (Taylor, 1990; Péra and Amrouz, 1998). Péra and Ambroise (1999) have calcined wastepaper sludge at 700–750°C to produce a mineral admixture for high-strength concrete composed of the resultant metakaolin and calcite. This admixture was as effective as a pozzolan as silica fume or metakaolin.

The variation in major oxide content (CaO, SiO_2, Al_2O_3 and MgO) of WSA from a major producer in the United Kingdom during a 5-year period Jun. 1996–May 2000 is illustrated in Fig. 22.2 (Kinuthia et al., 2001b). Over this period, the CaO content is

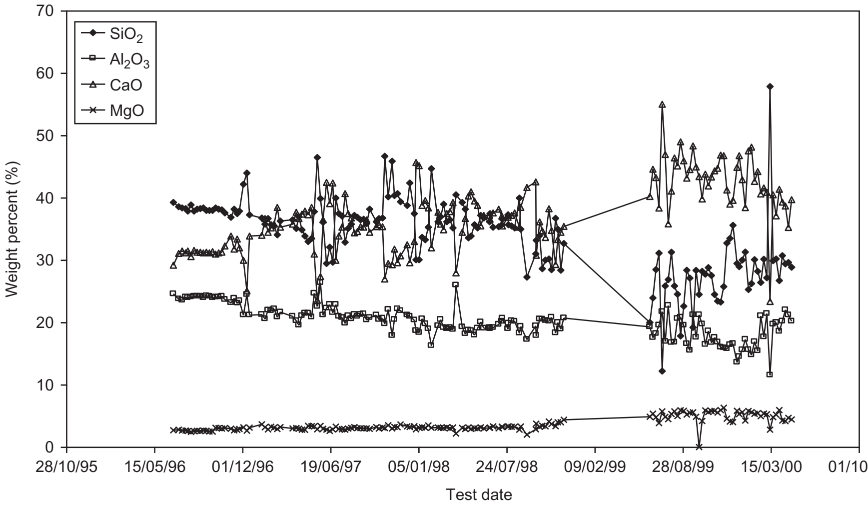

Fig. 22.2 Variation of major oxide component of wastepaper sludge ash with time (Kinuthia et al., 2001b).

seen to increase from about 30% to 45% and the SiO_2 content to decrease from about 40% to 25%. The Al_2O_3 content also decreased from about 25% to 20%, whilst the MgO content increased slightly to about 6%. The typical oxide content for this ash, as reported by various researchers, is illustrated in Table 22.2. The ash is known to be quite highly alkaline (pH 11–12) probably as a result of residual free lime.

The results in Table 22.3 show that the typical amount of soluble silica in WSA is about 23%, which indicates that of the silica present, a substantial proportion is reactive. There is also a high amount (mean 18%) of insoluble residue present in the

Table 22.2 Typical chemical % composition of WSA from a major UK producer

Oxide	O'Farrell et al. (2001)	Kinuthia et al. (2001b)	Bai et al. (2003)	Chaipanich et al. (2005)	Nidzam and Kinuthia (2011b)
CaO	43.31	42.71	43.51	43.51	36.82
SiO_2	24.21	24.17	25.70	25.70	33.9
Al_2O_3	18.70	18.39	18.86	18.86	18.05
MgO	5.18	5.04	5.15	5.15	5.44
Fe_2O_3	0.85	1.77	0.87	0.87	0.96
SrO	–	–	0.09	–	0.54
K_2O	–	–	1.31	–	0.46
TiO	–	–	0.68	–	0.37
P_2O_5	–	–	0.52	–	0.27
Na_2O	–	–	1.56	–	0.27
SO_3	1.12	1.08	1.05	1.05	0.24
MnO	–	–	0.04	–	0.03
BaO	–	–	0.04	–	0.024
CuO	–	–	–	–	0.0366
Li_2O	–	–	0.01	–	0.0041

Table 22.3 Chemical analysis data for wastepaper sludge ash (WSA)

Chemical composition	O'Farrell et al. (2001)	Kinuthia et al. (2001b)	Bai et al. (2003)
Insoluble residue	15.47	23.35	14.80
Soluble calcium oxide (CaO)	40.93	35.35	40.90
Soluble silica	22.83	23.95	22.55
Soluble Al_2O_3	15.37	–	15.60
Soluble MgO	2.29	–	2.30
Carbonate (CO_3)	3.10	2.09 (CO_2)	2.32
Soluble Fe_2O_3	0.66	–	0.66
Sulphate as SO_3	0.89	0.97	0.83

WSA, which is made up principally of anorthite and small amounts of gehlenite and quartz (detected with X-ray diffraction—XRD). These results, together with subsequent research, have suggested that some of the components may be pozzolanic in nature such as amorphous aluminosilicate phases, hydraulic such as calcium silicate/aluminate/aluminosilicate phases, or nonhydraulic crystalline phases such as anorthite and gehlenite. These properties of WSA are evidenced by its engineering properties in hydrated systems discussed in Section 26.6, which show the clearest evidence of the ability to set and to develop strength as a binder with and without any other additive or processing.

22.5.4 Mineralogical composition

The nonequilibrium treatment of the wastepaper sludge suggests that the resultant ash (WSA) may have a wide range of mineral components. The XRD analysis results of a dry WSA from a UK source was reported by Kinuthia et al. (2001b), Bai et al. (2003), Veerapan et al. (2003), Chaipanich et al. (2005) and Mozaffari et al. (2009) are shown in Fig. 22.3. The analysis shows that the crystalline phases present in the dry WSA were quartz, anorthite, gehlenite, vaterite/calcite and quicklime. The relative intensities of the various diffraction peaks suggest that, of the crystalline phases in the ash, gehlenite is the major phase present. In addition, metakaolin is commonly present, although due to its amorphous nature, it was not detected during XRD analysis. Frías et al. (2008) have also reported XRD work of a dry Spanish paper sludge waste, calcined at various temperatures within the range 600–800°C. They have corroborated the presence of kaolinite, quartz, calcite and lime, together with bicalcium silicate (C_2S).

22.5.5 Thermogravimetric (TG) analysis

Fig. 22.4 shows the derivative thermogravimetric (DTG) analysis of the dry WSA for temperatures of up to 1000°C. It shows a peak at 400–450°C, due to dehydroxylation of residual free lime (about 3–5% by weight) in the WSA. The broad peak at about

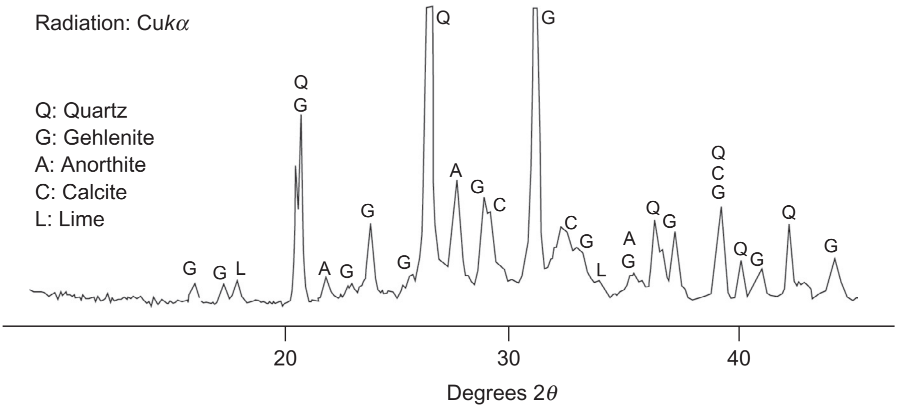

Fig. 22.3 XRD analysis of dry WSA.

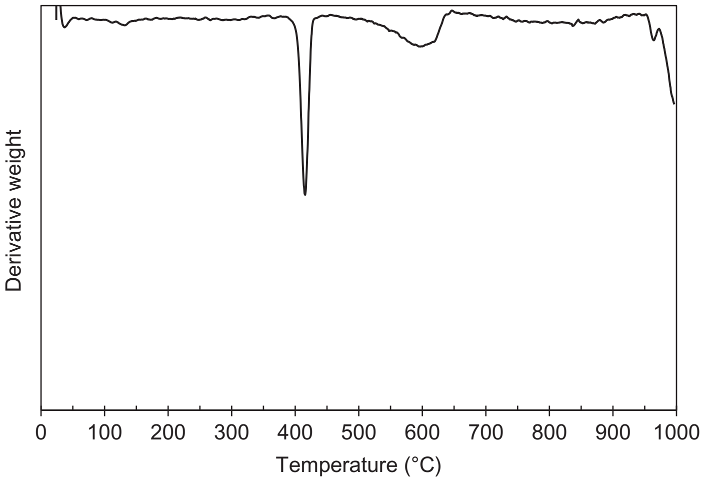

Fig. 22.4 Thermogravimetric (TG) analysis of WSA.

600°C is due to carbonate decomposition ($CaCO_3 \approx$ 1–2% by weight of WSA). The presence of carbonate is corroborated by the observation of calcite in XRD traces (Fig. 22.3) and of carbonate in the chemical analyses (CaO and CO_3 in Tables 22.2 and 22.3).

22.5.6 Potential for sustainable alternative cementation

Although WSA is much coarser than PC as already mentioned, its chemical and mineralogical composition, as well as TG analysis, both suggest the presence of a significant content of calcia. The chemical analysis further suggests that most of this calcia is soluble. It is therefore feasible that the aqueous reaction of this calcia with aluminosilicates would result in calcium-silicate, calcium-aluminate and calcium-aluminosilicate hydrates that would be expected to be very much similar to those present in hydrated PC systems. However, due to the coarse nature of the WSA, and to the lower temperature regimes prevailing during its production, the reactivity of WSA would be expected to be relatively lower than that commonly observed with PC. Utilization of all waste streams, including those occurring in significantly low volumes, are all contributory toward a futuristic zero industrial waste society (Kinuthia and Nidzam, 2011). In this regard, there are numerous other studies on the utilization of paper industry wastes as well other waste streams that are not in great abundance, such as Gemelli et al. (2001) and Vegas et al. (waste from paper industry); Khatib et al. (1997) and Wild et al. (1997; ground bricks); Khatib et al. (2013a, 2014; waste gypsum); and Mangat et al. (2006) and Khatib et al. (2008, 2013a, 2015; flue gas desulphurization—FGD waste). For this reason, as a waste and by-product material from the recycling of paper, any cementation shown by WSA has potential for sustainable utilization in construction. Such utilization will be the subject of this chapter, and will be pursued in the remaining sections of the chapter.

22.6 Properties and utilization of wastepaper sludge ash (WSA) in construction

Most paper is normally coated with clay and limestone to create the familiar smooth surfaces. When paper is recycled, the clay and limestone remain in the sludge from which the recyclable cellulose fibrous material is removed during the reprocessing. When this sludge is combusted to reduce volume of landfilled waste, the clay and limestone are heated in the process. Combusting wastepaper sludge inadvertently produces a waste material that has undergone more or less a similar heat process as PC, albeit at lower temperatures. For many years, this WSA material has ended up in landfill, without the knowledge that this is similar to a very weak cement. There have been numerous attempts to utilize this wastepaper-based weak cement in construction. Nearly all documented attempts involve the use of WSA. There have been relatively few attempts to use uncombusted wastepaper, either in paper or dewatered sludge form (Ahmadi and Al-Khaja, 2001). This chapter will mainly dwell on the utilization of combusted paper sludge waste to produce WSA. The coverage will include both the applied research on WSA-based cementitious binders, progressively leading to actual applications in building and construction.

22.6.1 Properties of WSA-based binders

It has already been suggested that WSA has shown ability to set and develop strength. Initial work by Kinuthia et al. (2001b) has shown that WSA on its own not only sets far too quickly (within about 10 min) but also develops low strength that is not likely to find any meaningful application. For this reason, WSA has commonly been used in combination with other hydraulic or latently hydraulic materials, such as PC and/or GGBS. PC is a hydraulic material that is well known, and its performance as a binder is well established. Its partial replacement using WSA is therefore likely expected to be successful. It is more a question of the effect of WSA on the already well-established good performance of PC. Partial replacement of PC with WSA has been reported by O'Farrell et al. (2001) and Frías et al. (2008). The former used a UK-produced WSA and a wide range of replacement levels, while Frías et al. (2008), using WSA sourced from Spain, reported on only one replacement level of 10%. For both UK- and Spanish-based WSA sources, the blended binder was found to possess enhanced compressive strength development. The work on UK-based WSA predominantly involved blended use of (i) WSA with PC and/or GGBS in mortar and concrete, or (ii) WSA with lime and/or GGBS in soil-based materials. These two broad approaches have enabled the development of a range of 'green' cements for masonry, concrete and soil-based materials. The WSA-based binders have performed very well in terms of strength and durability, and for concrete, showed what many users regarded a better light-coloured appearance compared with the traditional PC. The work involved in the research for product development typically starts with the relevant oxide, chemical and other analytical considerations of the individual constituents, in order to facilitate a holistic and informed optimization of material combinations that will lead to robust applications.

22.6.2 Chemical analyses

The oxide and chemical analyses of WSA have already been presented in Tables 22.2 and 22.3. In most published applications, the ash has been used in combination with the more established traditional binders (lime and PC) and/or with the currently major industrial by-product materials such as GGBS. Table 22.4 shows a comparison of the oxide composition of WSA with the equivalent compositions for two of the materials with which WSA has been more commonly effectively combined—GGBS and PC.

22.6.3 Thermogravimetric analyses

The reasoning behind the matching of WSA with GGBS is that GGBS requires only a small amount of lime, such as the free lime that is present in WSA, to unlock potential for further and enhanced hydration. Kinuthia et al. (2001b) started this endeavour by analyzing the TG profiles on hydrated blended mixtures of WSA and GGBS. Fig. 22.5 shows the DTG analyses of dry (unhydrated) WSA: GGBS blends for temperatures of up to 1000°C.

All the blends shown in Fig. 22.5 show the peak at 400–450°C, observed earlier from the dehydroxylation of hydrated lime in the WSA as well as the broad peak at about 600°C due to carbonate decomposition. GGBS shows no majors peaks when heated up to 1000°C. Thus, both peaks (for lime and carbonate) are progressively reduced by the increasing proportion of GGBS in the WSA–GGBS blends.

The DTG analyses of the hydrated WSA–GGBS blends in Fig. 22.5 cured in water for up to 90 days were carried out by Kinuthia et al. (2001b) and are shown in Fig. 22.6. The lime peaks are absent and have been noted to disappear as early as

Table 22.4 Oxide composition and physical properties of GGBS and portland cement

	Composition (%)		
Oxide	**WSA**[a]	**GGBS**[b]	**PC**[c]
CaO	41.59	42.0	65.6
SiO_2	27.00	35.5	21.0
Al_2O_3	18.50	12.0	4.63
MgO	5.20	8.0	1.18
Fe_2O_3	1.11	0.4	2.26
SO_3	0.87	0.2	2.69
Insoluble residue	–	0.3	0.30

a Average from Table 22.1.
[b] Provided by Civil and Marine Slag Cement, UK.
[c] Provided by Rugby Cement, UK.
Kinuthia, J.M., O'Farrell, M., Sabir, B.B., Wild, S., 2001b. A preliminary study of the cementitious properties of wastepaper sludge ash (WSA)–ground granulated blast-furnace slag (GGBS) blends. In: Dhir, R.K., Limbachiya, M.C., Newlands, M.D. (Eds.), Proceedings of the International Symposium on Recovery and Recycling of Paper, Dundee University, 19th March 2001, Thomas Telford, pp. 93–104, ISBN: 0-7277-2993.

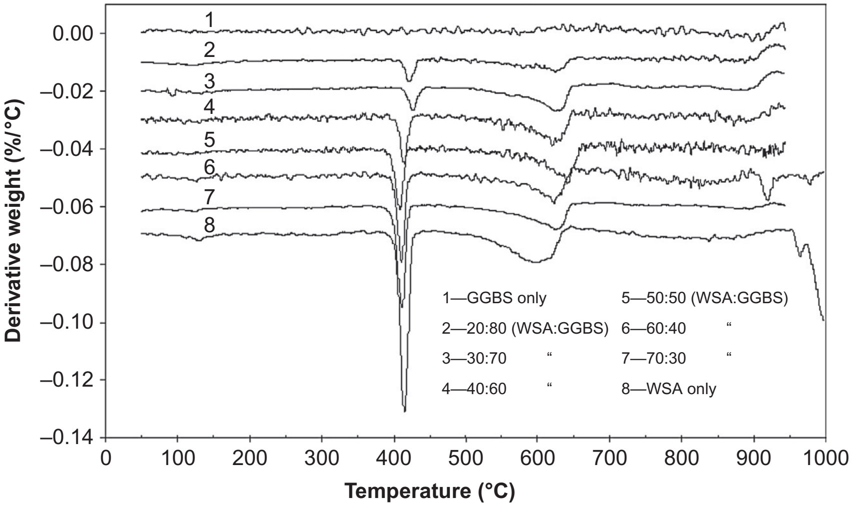

Fig. 22.5 Thermogravimetric analysis (TGA) of the dry WSA: GGBS blends (Kinuthia et al., 2001b).

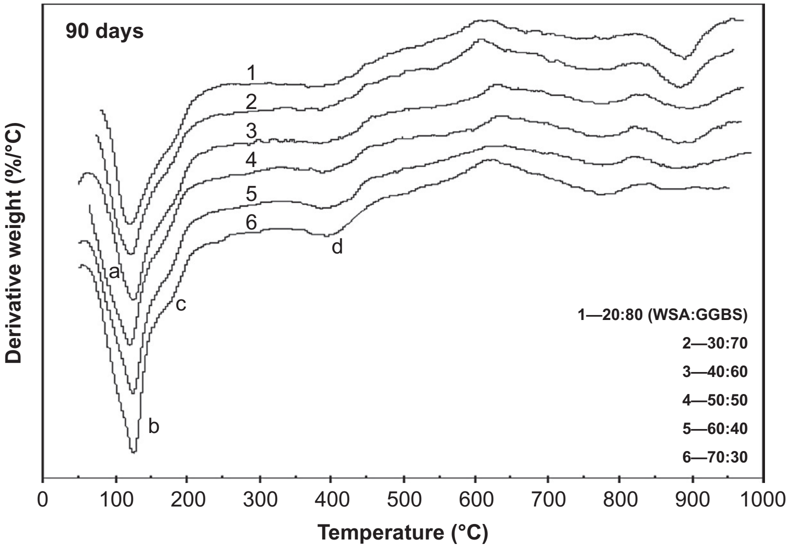

Fig. 22.6 Thermogravimetric analysis (TGA) of hydrated dry WSA–GGBS blends (Kinuthia et al., 2001b).

at 3 days of hydration (not shown). They are replaced by peaks (a, b and c) in the range of 70–200°C. Both WSA and GGBS contain some sulphate, 0.97% and 0.2%, respectively, thus besides C–S–H gel, the peaks (a and b) are also likely to be due to the decomposition of ettringite. At the high WSA contents (plots 5—60:40 and 6—70:30 WSA–GGBS), a third (shoulder) peak (c) is observable at about 180°C that increases with increasing WSA content, and is thought to be due either to formation of gehlenite hydrate or a carboaluminate hydrate. Between 28 and 90 days a further peak (d) develops at 350–380°C, which may be due to hydrogarnet formation (Kinuthia et al., 2001b).

22.6.4 Setting time

O'Farrell et al. (2001) determined both initial and final setting times of various PC–WSA binder blends using the Vicat apparatus (as described in BS EN196 Part 3). The results obtained are illustrated in Fig. 22.7. They demonstrate that the presence and amount of WSA in the blend has a profound effect on the paste properties. Increasing WSA content at the expense of PC reduced substantially both the initial and final set of the pastes. Perhaps the most significant trend observed is that of the reduction in time for the final set to occur. As the amount of WSA increases (and therefore the amount of retardation increases), the period between initial and final set shows a marked reduction. For example, at replacement levels of 40% and 60%, the difference between initial and final set is 10 and 5 min, respectively.

With the addition of GGBS in the PC–WSA, the initial setting time is marginally increased when compared with equivalent replacement of PC by WSA alone (Fig. 22.8). The initial set continues to decrease, however, with increasing cement replacement by the 50WSA:50GGBS blend. The more significant effect of the inclusion of GGBS is the reversal of the trend observed for the final set. Final settings times increase with increasing GGBS content. At low PC contents (ie, 60% and 80% replacement), the time required before the final set has occurred increases to over 1 day for 60% replacement and over 3 days for 80% replacement. Similarly, long final setting times have been observed by Khatib et al. (2013a,b, 2014, 2015), using slag–fly ash gypsum blends, which show delayed setting time up to 3 days. Concrete made with these two particular blends experienced deterioration when introduced to the water-curing environment. The results indicate the importance of achieving final set for the integrity of concrete. The concrete made with the 40-30-30 blend

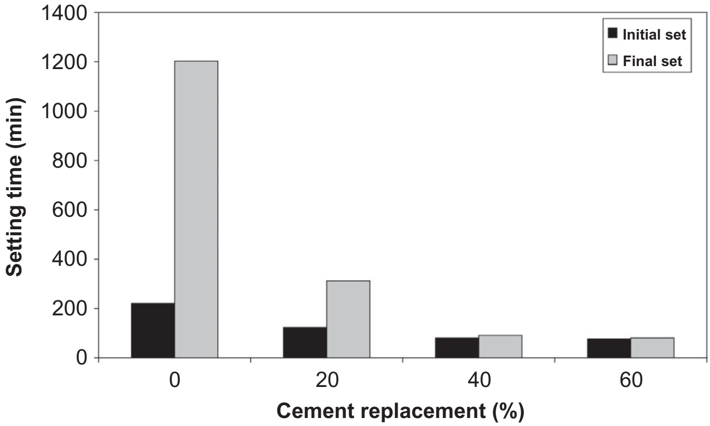

Fig. 22.7 Setting times of PC–WSA mixtures (O'Farrell et al., 2001).

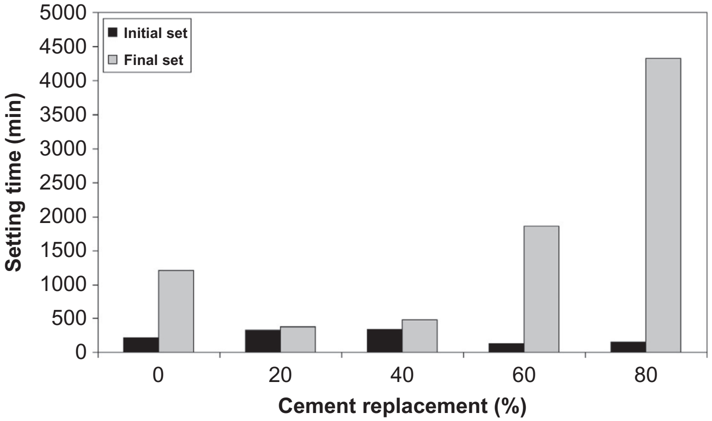

Fig. 22.8 Setting times of PC-(50% WSA–50% GGBS) mixtures.

(PC–WSA–GGBS, ie, 60% PC replacement) showed signs of disintegration after 1 day in the mould but no signs of distress if demoulded after 3 days. The final set of this blend occurred within the intervening period in the prolonged containment in the mould, after about 31 h (1860 min). The concrete made with the 20-40-40 blend still exhibited deterioration after 3 days in the mould and the final set for the paste made with this blend was not observed even after 3 days.

22.6.5 Compressive strength

22.6.5.1 Mortar

Table 22.5 shows the compressive strengths of WSA–GGBS mortars, water cured and moist cured for up to 28 days (Kinuthia et al., 2001b). The equivalent strength values for PC mortar at the same age are also shown. At 1 day, the compressive strength values are very low but as curing periods increase to 28 days, the strengths increase significantly relative to those of PC mortar, indicating substantial cementing potential of WSA–GGBS blended binders. In particular, water-cured mortar with a binder composition of 60:40 achieves a 28-day strength (15.0 MPa) in excess of 50% of the strength of the control PC mortar (28.8 MPa). In general, the strengths of moist-cured mortars are lower than the strengths of equivalent water-cured mortars. Mortars with a high WSA content such as the 60:40 (WSA–GGBS) binder composition showed much lower strength values when moist cured as opposed to water cured. This is thought to be due to the high water demand during the hydration of WSA hydration compared with that for GGBS.

22.6.5.2 Concrete

The research work on WSA–GGBS mortar was extended to concrete, in a comprehensive study on the compressive strengths of WSA:GGBS concrete by Kinuthia et al. (2001a,b). The work on concrete did not adopt 20:80 (WSA–GGBS) blended binder that was observed to result in weak mortar. The work however extended the frontier

Table 22.5 Strength of WSA:GGBS mortar (MPa) (relative strengths in parentheses)

		Mix composition (WSA:GGBS)					
Curing period (days)	**Curing regime**	**20:80**	**30:70**	**40:60**	**50:50**	**60:40**	**PC**
1	Moist	0.2 *(0.02)*	0.7 *(0.06)*	1.1 *(0.10)*	1.2 *(0.11)*	0.5 *(0.04)*	11.6
	Water	0.2 *(0.02)*	1.0 *(0.09)*	1.4 *(0.13)*	1.4 *(0.13)*	1.5 *(0.14)*	11.1
7	Moist	4.8 *(0.22)*	5.8 *(0.26)*	6.9 *(0.31)*	7.6 *(0.35)*	3.7 *(0.17)*	22.1
	Water	3.9 *(0.19)*	6.2 *(0.30)*	7.7 *(0.38)*	7.8 *(0.38)*	8.9 *(0.43)*	20.5
28	Moist	12.2 *(0.40)*	12.7 *(0.41)*	12.1 *(0.39)*	12.0 *(0.39)*	9.5 *(0.31)*	30.8
	Water	12.1 *(0.42)*	13.0 *(0.45)*	12.4 *(0.43)*	13.1 *(0.45)*	15.0 *(0.52)*	28.8

Kinuthia, J.M., O'Farrell, M., Sabir, B.B., Wild, S., 2001b. A preliminary study of the cementitious properties of wastepaper sludge ash (WSA)–ground granulated blast-furnace slag (GGBS) blends. In: Dhir, R.K., Limbachiya, M.C., Newlands, M.D. (Eds.), Proceedings of the International Symposium on Recovery and Recycling of Paper, Dundee University, 19th March 2001, Thomas Telford, pp. 93–104, ISBN: 0-7277-2993.

of WSA utilization beyond 60:40 WSA–GGBS, by including a new formulation using 70:30 WSA–GGBS. This was with a view to pursuing more uptake of the WSA in the concrete, based on the strong performance on the compressive strength of the 60:40 blended mortar as seen earlier. The work on concrete was carried out at w/b ratios of 0.5 and 0.4, and is summarized in Table 22.6 and graphically illustrated in Fig. 22.9. In the work, PC concrete was used as control concrete. It is seen that beyond 1 day, the concrete with the 50:50 binder compositions at both w/b ratios develops the highest strength (shown in bold). Further increases in WSA–GGBS ratio give lower strengths. For binder compositions up to 50:50, lowering the w/b ratio from 0.5 to 0.4 generally produces increased strength although the advantage of the lower w/b ratio decreases with increasing WSA–GGBS ratio. Above a 50:50 composition ratio, the strengths of concrete with w/b ratio 0.4 decline more rapidly with increase in WSA–GGBS ratio than do those for concrete with w/b ratio 0.5. Thus, at composition 70:30 the strengths at w/b ratio 0.4 are substantially less than at w/b ratio 0.5. It is suggested that this is a result of the much higher water demand and poorer compaction at high WSA–GGBS ratios.

The strength of concrete made with WSA:GGBS as binder is substantially below that made with PC (Table 22.6), the former achieving the highest relative strength value of 0.42 for the 50:50 blend. Also the rate of early strength development is much slower for the WSA:GGBS concrete. For example, at 1 day concrete made with PC achieved about 52% (w/b ratio 0.5) and 72% (w/b ratio 0.4) of its strength at age 28 days, while concrete with WSA:GGBS binders achieved only 2–6% of the 28-day strength for both w/b ratios. However, the strength of WSA:GGBS concrete shows substantial improvement between 1 and 28 days and continues to show significant increase up to 90 days.

Having achieved maximum relative strength of only about 0.42 of the control PC concrete using WSA–GGBS formulations, work by O’Farrell et al. (2001) investigated the effect of incorporating PC in the WSA–GGBS concrete. The research work on compressive strength PC:WSA and PC–WSA:GGBS concrete cured for up to 90 days is summarized in Table 22.7 and in Fig. 22.10. The strength values of the ternary blended binders composed of PC–WSA–GGBS are much higher compared with those reported earlier using the WSA–GGBS binary blended binder. Fig. 22.10A shows the compressive strength values of the PC:WSA system concrete, expressed relative to the control concrete (where binder is 100% PC). The presence of WSA as a partial cement replacement has, at early ages, the effect of decreasing compressive strength. This decrease in compressive strength increases with increasing WSA content. At the early curing times (7 days), all replacement levels result in compressive strength values that are lower than that of the control concrete and increasing replacement results in lower compressive strength. However, as the curing period increases to 90 days, the compressive strength reaches, and in all cases (up to 60% total replacement) exceeds, that of the control. The trend of strength gain of the PC–WSA and PC–WSA–GGBS systems indicates that these mixtures, after initially showing low compressive strength, undergo additional cementing reactions leading to increased rates of strength gain and ultimately resulting in elevated compressive strengths when compared to concrete with a 100% PC binder.

Table 22.6 Strength of WSA–GGBS concrete (MPa) at w/b ratios of 0.5 and 0.4 (relative strengths in parentheses; strongest mix compositions beyond 1 day of curing are shown in bold)

		Mix composition (WSA:GGBS)					
Curing period (days)	**w/b ratio**	**30:70**	**40:60**	**50:50**	**60:40**	**70:30**	**PC**
1	0.5	0.6 *(0.02)*	0.4 *(0.01)*	0.6 *(0.02)*	0.6 *(0.02)*	1.2 *(0.04)*	30.5
	0.4	0.7 *(0.01)*	1.0 *(0.02)*	1.3 *(0.02)*	0.8 *(0.02)*	1.5 *(0.03)*	52.3
7	0.5	10.8 *(0.21)*	8.6 *(0.17)*	**11.0 *(0.21)***	7.7 *(0.15)*	10.5 *(0.21)*	51.1
	0.4	10.2 *(0.16)*	9.6 *(0.15)*	**11.9 *(0.18)***	4.0 *(0.06)*	5.6 *(0.09)*	64.9
28	0.5	16.8 *(0.29)*	16.3 *(0.28)*	**19.0 *(0.33)***	15.2 *(0.26)*	16.9 *(0.29)*	58.1
	0.4	18.0 *(0.24)*	18.9 *(0.25)*	**20.7 *(0.28)***	16.4 *(0.22)*	12.5 *(0.17)*	75.1
90	0.5	22.7 *(0.29)*	24.0 *(0.30)*	**27.1 *(0.34)***	23.4 *(0.29)*	22.7 *(0.28)*	79.8
	0.4	26.8 *(0.41)*	26.2 *(0.40)*	**27.0 *(0.42)***	22.7 *(0.35)*	17.5 *(0.27)*	65.0

Kinuthia, J.M., Gailius, A., Laurikietytė, Ž., 2001a. Compressive strength and workability of concrete utilising waste-paper sludge ash and ground granulated blast furnace slag as binder. Paper Presented at the 7th International Conference on Modern Building Materials, Structures and Techniques, Vilnius Gediminas Technical University, Vilnius, Lithuania, 16–18th May 2001; Kinuthia, J.M., O'Farrell, M., Sabir, B.B., Wild, S., 2001b. A preliminary study of the cementitious properties of wastepaper sludge ash (WSA)—ground granulated blast-furnace slag (GGBS) blends. In: Dhir, R.K., Limbachiya, M.C., Newlands, M.D. (Eds.), Proceedings of the International Symposium on Recovery and Recycling of Paper, Dundee University, 19th March 2001, Thomas Telford, pp. 93–104, ISBN: 0-7277-2993.

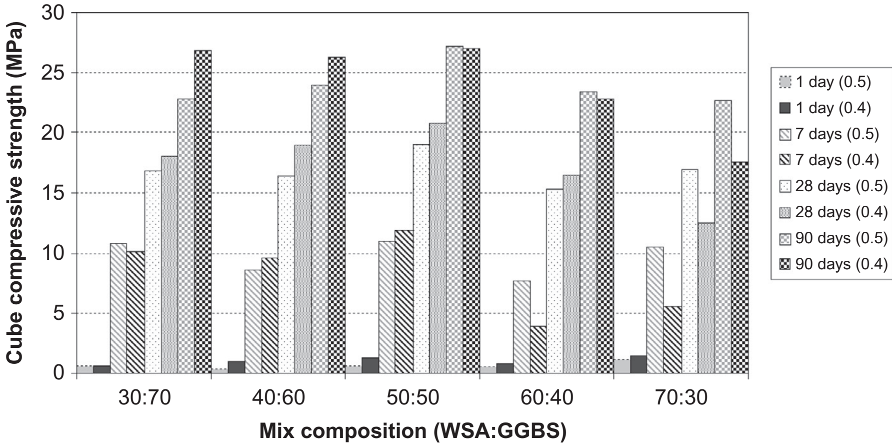

Fig. 22.9 Strength of WSA–GGBS concrete at w/b ratios of 0.5 and 0.4 (Kinuthia et al., 2001a,b).

Table 22.7 Compressive strengths (N/mm^2) of concrete with PC–WSA and PC-(50% WSA–50% GGBS) binder

Mix (ingredients)	Mix (proportions)	Curing period (days)		
		7	28	90
PC control	100-0-0	45.4	56.0	59.5
PC–WSA	80-20-0	43.2	57.0	67.9
	60-40-0	42.4	58.7	67.8
	40-60-0	28.8	45.2	61.5
PC–WSA–GGBS	80-10-10	44.7	57.7	68.0
	60-20-20	40.9	59.1	68.2
	40-30-30	32.9	57.4	64.4

O'Farrell, M., Chaipanich, A., Kinuthia, J.M., Sabir, B.B., Wild, S., 2001. A new concrete incorporating wastepaper sludge ash (WSA). In: Dhir, R.K., Hewlett, P.C., Csetenyi, L.J. (Eds.), Innovations and Developments in Concrete Materials and Construction, Thomas Telford, pp. 149–158, ISBN: 0-7277-3179-3.

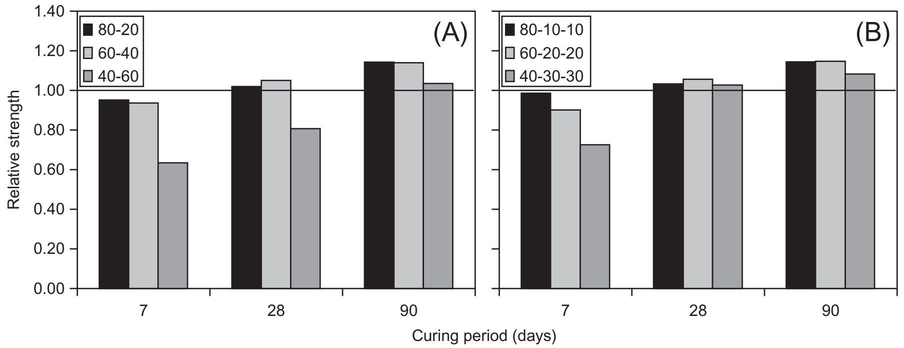

Fig. 22.10 Relative compressive strength of concrete with (A) PC–WSA binder and (B) PC-(50% WSA–50% GGBS) binder (O'Farrell et al., 2001).

At lower WSA contents (up to 40% replacement), the drop in compressive strength is less than 10%. As curing times increase to 90 days, all the compressive strengths of the WSA-containing concrete are all in excess of those obtained for the control concrete. This is including concrete with binders where 60% of the PC is replaced by WSA, although there is still a general trend of decreasing strength with increasing WSA content. For replacement levels up to 40%, the compressive strengths are about 115% of those of the control. A similar trend is observed for those concretes whose binder is part of the PC–WSA–GGBS system (see Fig. 22.9B), although in this, the control concrete is exceeded by all WSA-containing concrete at an earlier curing age of 28 days.

22.6.5.3 Masonry

Research work by Veerapan et al. (2003) utilized the results of WSA–GGBS binders developed by Kinuthia et al. (2001a,b) and PC–WSA–GGBS binders developed by O'Farrell et al. (2001) to produce concrete masonry blocks for building and other construction. The laboratory work culminated in pilot trials composed of (1) trial production using a commercial block manufacturer and (2) trial construction. Fig. 22.11 shows blocks manufactured at various moisture contents during commercial optimization for maximum strength and durability. These blocks were produced using an automated process similar to that adopted in full commercial production. Fig. 22.12 shows two pilot block masonry walls, one made using blocks made of WSA–GGBS binder, side by side with a control wall of typical blocks made using PC. The novel WBS-GGBS blocks were as easy to handle as their PC-based counterparts, and some users found the marginally lighter colour more attractive.

22.6.5.4 Compressed earth

The success of WSA applications in soil-based construction has been demonstrated by Nidzam and Kinuthia (2010, 2011a). Fig. 22.13 shows results of tests on unconfined compressive strength of Lower Oxford Clay (LOC) soil (of UK) stabilized with lime in

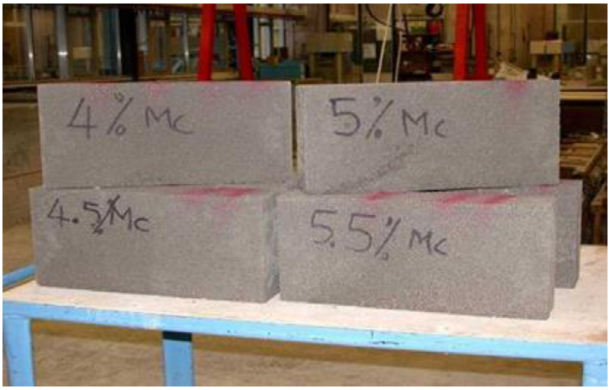

Fig. 22.11 Concrete blocks made with wastepaper sludge ash (WSA; Veerapan et al., 2003).

Fig. 22.12 Demonstration of two concrete block walls that showcase the utilization of wastepaper sludge ash (WSA) in the development of ‘green’ cement.
Courtesy: Author.

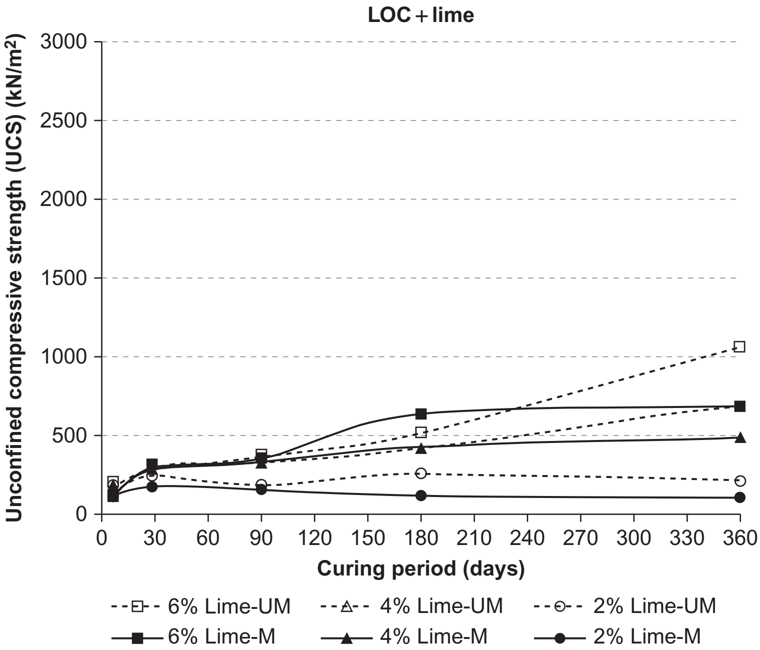

Fig. 22.13 Unconfined compressive strength of LOC stabilized with lime in both mellowed and unmellowed conditions (Nidzam and Kinuthia, 2011a).

both mellowed and unmellowed conditions. For road construction, two curing regimes of mellowing and unmellowed stabilized states may be used. Mellowing is a particularly British practice adopted during road construction using stabilized soil, whereby the stabilized material is allowed to cure ('mellow') for a period of a maximum of 3 days, to allow the stabilizer (especially lime) to achieve maximum modification of the soil before final compaction. Fig. 22.14 shows the same LOC soil stabilized using WSA. The results using WSA show better performance relative to lime stabilization and also that with both stabilizers the unmellowed condition is preferable. Nidzam and Kinuthia (2011a) attributed the poorer performance of the mellowing practice to the consumption (or 'fixing') of the critically needed calcium ions in the lime and in the WSA during the mellowing stages, and thus compromising the formation of the C–S–H gel pozzolanic binder during subsequent solidification of the material during prolonged curing.

Fig. 22.15 shows unconfined compressive strength when the LOC in Figs 22.13 and 22.14 was stabilized with a WSA–GGBS blended binder at 50:50 WSA:GGBS ratio, in both mellowed and unmellowed conditions. It is clear that GGBS significantly enhances the performance of WSA as a soil stabilizer in a similar manner to that seen earlier using mortar and concrete. It will be seen later that the enhancement in performance is not limited to strength but goes further to include enhanced volume stability upon successful suppression of swelling potential, when the WSA–GGBS blended binder is used on the extremely expansive system of lime stabilization of the sulphate-bearing LOC.

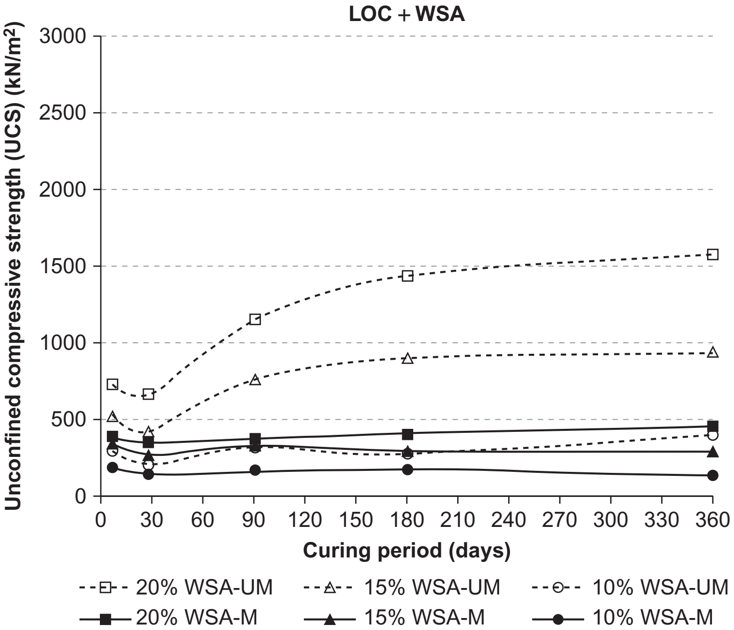

Fig. 22.14 Unconfined compressive strength of LOC stabilized with WSA in both mellowed and unmellowed conditions (Nidzam and Kinuthia, 2011a).

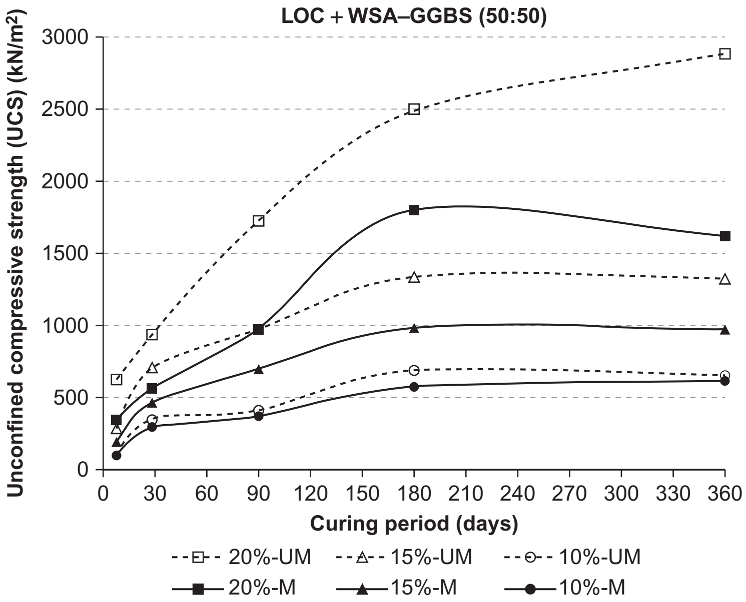

Fig. 22.15 Unconfined compressive strength of LOC stabilized with two WSA–GGBS blends at 50:50 WSA:GGBS ratio in both mellowed and unmellowed conditions (Nidzam and Kinuthia, 2011a).

22.7 Durability of WSA-based building and construction materials

22.7.1 Compressive strength

Kinuthia et al. (2001b) researched the compressive strengths of WSA–GGBS mortars, water cured for 28 days and then exposed to either sodium sulphate solution or water for 142 days as shown in Table 22.8. Little difference is apparent between the

Table 22.8 **Strength of WSA:GGBS mortar (MPa) cubes exposed to sodium sulphate solution or deionized water for 142 days (relative strengths in parenthesis)**

Curing regime	Mix composition (WSA:GGBS)					
	20:80	**30:70**	**40:60**	**50:50**	**60:40**	**PC**
Sulphate	21.6 *(0.86)*	20.4 *(0.81)*	17.6 *(0.70)*	15.2 *(0.60)*	13.6 *(0.54)*	25.2
Water	23.2 *(0.87)*	21.2 *(0.79)*	16.4 *(0.61)*	18.0 *(0.67)*	14.8 *(0.55)*	26.8

Kinuthia, J.M., O'Farrell, M., Sabir, B.B., Wild, S., 2001b. A preliminary study of the cementitious properties of wastepaper sludge ash (WSA)—ground granulated blast-furnace slag (GGBS) blends. In: Dhir, R.K., Limbachiya, M.C., Newlands, M.D. (Eds.), Proceedings of the International Symposium on Recovery and Recycling of Paper, Dundee University, 19th March 2001, Thomas Telford, pp. 93–104, ISBN: 0-7277-2993.

strengths of mortar cubes exposed to sodium sulphate solution and those stored in water. However, testing took place at the exposure period when the PC mortar had just begun to show significant expansion and the WSA–GGBS mortars had shown no expansion. If testing had been delayed for a much longer period, then it is anticipated that the sulphate exposed PC mortar would have shown much reduced strength or collapsed. The strength of the mortar exposed to both sodium sulphate solution and in water decreases with increasing WSA content. It is interesting to note that this trend is opposite that observed for mortar cubes cured in water and tested in compression at 28 days (see earlier in Table 22.5). Also the strengths relative to PC mortar are much higher after 142 days than at 28 days particularly for the high GGBS:WSA ratio blended mortars. This behaviour indicates the increasing influence, with age, of GGBS on WSA–GGBS mortar strength, and suggests that the WSA contributes significantly to strength between 1 and 28 days, and the GGBS contributes significantly to strength beyond 28 days. In addition, poorer compaction at high WSA:GGBS ratios reduced the strength development of these mortars although the higher w/b ratio (0.65) of the mortars means that this is much less of a problem with the mortars than with the concretes (w/b 0.4 and 0.5).

22.7.2 Volume stability

The volume stability of the WSA–GGBS mortar mix compositions used for the cube tests in Section 26.7.1 was assessed by analyzing the linear expansion of mortar prisms. This was carried out with the prisms being exposed to sodium sulphate solution, and this was undertaken for up to 250 days. The results are plotted in Fig. 22.16, which shows that after 112 days of exposure, the control (ie, PC) mortar prisms showed rapid

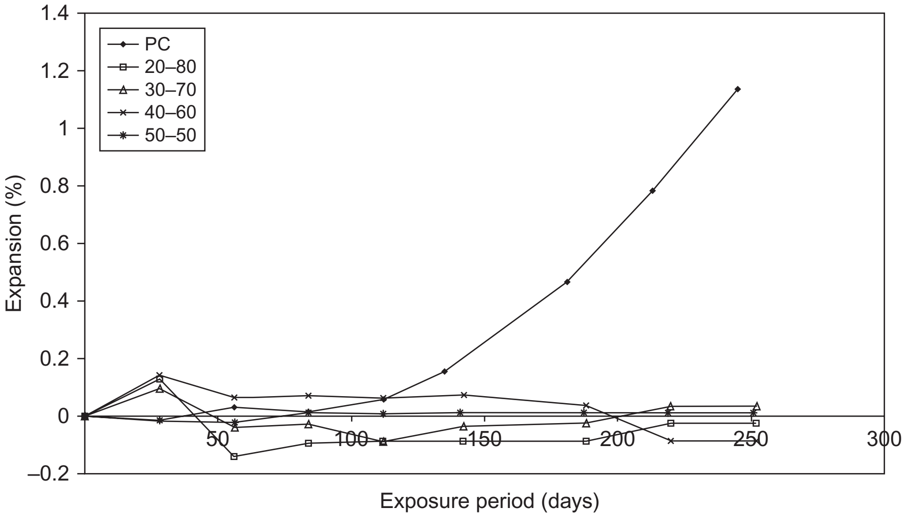

Fig. 22.16 Expansion of WSA:GGBS mortars exposed to sodium sulphate solution (Kinuthia et al., 2001b).

increase in linear expansion. This is a relatively short time period prior to the onset of expansion of the PC mortar. During this period of soaking in a sulphate solution, the WSA–GGBS mortar prisms, regardless of composition, showed no significant expansion nor were there any visible signs of deterioration.

The robustness of the WSA and WSA–GGBS sustainable binders was again demonstrated by the volume stability of the soil–WSA–GGBS system illustrated in Figs 22.17 and 22.18. These figures show the successful work by Nidzam and Kinuthia (2011a), who managed to suppress the swelling potential by using WSA–GGBS blended binder on an extremely expansive process that is well documented, that of lime stabilization of sulphate-bearing soils such as LOC. In both figures, the test specimens were moist cured in sealed (humid) Perspex chambers for 7 days before partially soaking them in water until maximum linear expansion was observed.

Overall, the research results overwhelmingly demonstrate that the utilization of WSA in building and in general construction is feasible, and especially so upon combination with carefully selected other compatible materials, sometimes industrial waste and/or by-product materials, so as to achieve an extremely high level of sustainability and care for the built environment.

22.8 Future trends

Although the paper industry has a long manufacturing history, the recycling of paper has a relatively short history. For this reason, the application of wastepaper in construction is a relatively new concept that is likely to grow in the coming years. The work reported in this chapter is predominantly that associated with the author of

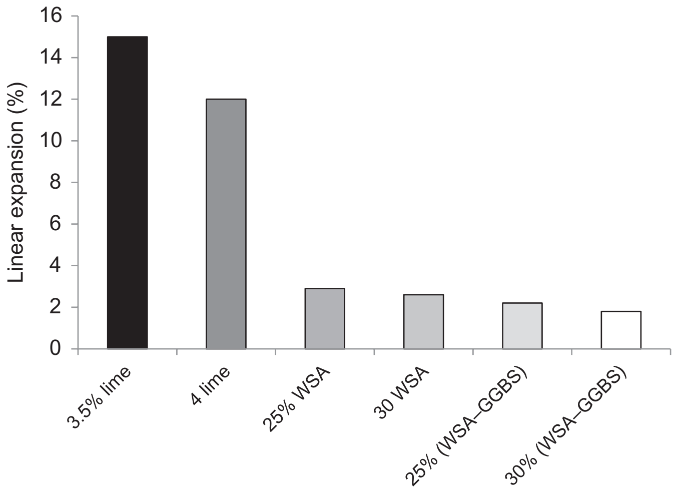

Fig. 22.17 Significant reduction in the maximum linear expansion of an expansive sulphate-bearing clay soil system by the combined use of wastepaper sludge ash (WSA) and ground granulated blast furnace slag (GGBS; Nidzam and Kinuthia, 2011a).

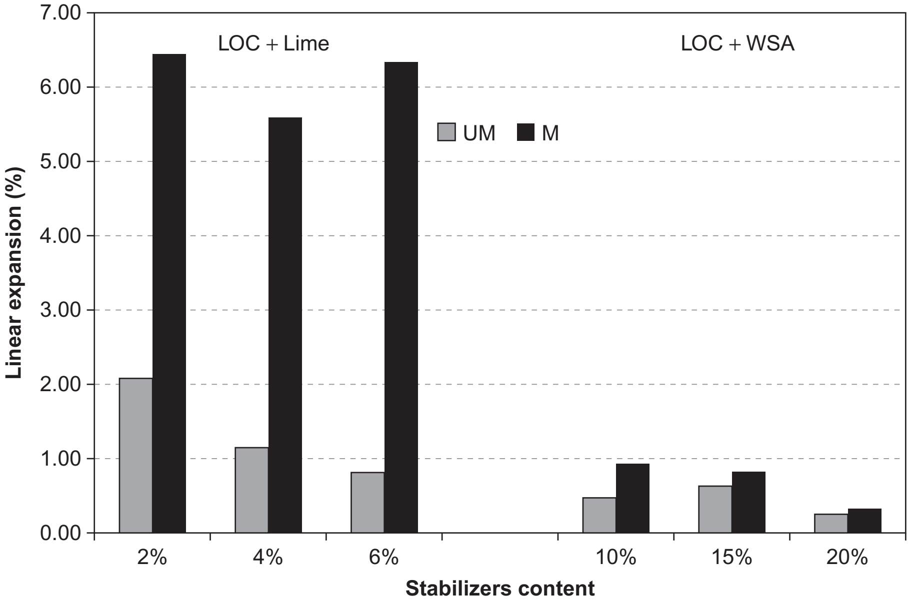

Fig. 22.18 Maximum linear expansion upon soaking of unmellowed (UM) and mellowed (M) lime-stabilized LOC and WSA-stabilized LOC (Nidzam and Kinuthia, 2011a).

this chapter (on WSA), supported by coworkers. There is a growing body of work, however, from other researchers who have made attempts to utilize wastepaper in both cementitious and noncementitious contexts. In the cementitious context, some researchers have focused on the binder aspect, especially utilizing WSA, while others have focused on cementitious involvement that does not involve the combusted or incinerated wastepaper. Table 22.9 classifies some of the work known to the author, along these considerations.

Most researchers have established that it is possible to partially replace PC with ash from the combustion or the incineration of wastepaper, with or without working in combination with other supplementary additives such as GGBS, to produce effective binders for paste, mortar, concrete, cement-based masonry and soil-based masonry, and other construction materials. In some cases, it has been possible to achieve very high levels of PC replacement, and in some rarer cases total replacement, as was seen with WSA–GGBS blended binders. Some formulations achieved compressive strengths in excess of the control materials made using either lime or PC.

For cementitious systems, most of the strength development is hampered by the property of wastepaper ash of very high water demand. The use of retarders/plasticizers is essential at replacement levels in excess of 20% wastepaper ash, in order to achieve mixes of workability similar to that of the control. The use of such admixtures does have a significant influence on the behaviour of the setting times of these mixtures, however, and it is vital that final set be achieved before water-curing commences. The work carried out using wastepaper has synergy with that ongoing

Table 22.9 Classification of some studies on wastepaper and allied wastes/additives

Cementitious	Cementitious	Noncementitious	Remarks
Ahmadi and Al-Khaja (2001)	✓	✓	Paper waste sludge
Aigbomian and Fan (2013)	✓		Wastepaper
Kinuthia et al. (2001a,b), Bai et al. (2002, 2003), Veerapan et al. (2003), Chaipanich et al. (2005) and Mozaffari et al. (2009)	✓		Wastepaper sludge ash (WSA); Wastepaper sludge ash–ground granulated blast furnace slag (WSA–GGBS)
Banfill and Frias (2007)	✓		Calcined paper sludge
Celebi et al. (2005)		✓	Wastepaper
Conesa et al. (2008)	✓		Paper wastes
Rajput et al. (2012)		✓	Cotton and recycle paper mill waste
Dunster (2007)	✓		Paper sludge and paper sludge ash
Eren et al. (1995)			Fly-ash and slag cement
Frederick et al. (1996)		✓	Recycled paper sludge
Frías et al. (2008)	✓		Art paper sludge waste
Gemelli et al. (2001)	✓	✓	Paper industry wastes
Nidzam and Kinuthia (2010)	✓		Kaolinite clay and Lower Oxford Clay with: Wastepaper sludge ash (Clay–WSA); Wastepaper sludge ash–ground granulated blast furnace slag (Clay–WSA–GGBS)
Nidzam and Kinuthia (2011a,b)	✓		Lower Oxford Clay–lime (LOC–L); LOC–wastepaper sludge ash (LOC–WSA); LOC-wastepaper sludge ash-lime (LOC–WSA–L); LOC–wastepaper sludge ash–ground granulated blast furnace slag (LOC–WSA–GGBS)
Nidzam and Kinuthia (2011b)	✓		LOC–wastepaper sludge ash–portland cement (LOC–WSA–PC);

Continued

Table 22.9 Continued

Cementitious	Cementitious	Noncementitious	Remarks
O'Farrell et al. (2001) and Veerapan et al. (2003)	✓		Portland cement–wastepaper sludge ash (PC–WSA); Portland cement–wastepaper sludge ash–ground granulated blast furnace slag (PC–WSA–GGBS)
Okeyinka and Idowu (2014)		✓	Paper waste–ceiling boards
Péra and Ambroise (1998)	✓		Paper sludge
Péra and Ambroise (1999)	✓		Paper sludge
Péra and Ambroise (1998)	✓		Paper sludge
Ridzuan et al. (2011)			Wastepaper sludge ash (WSA)
Salvador (1995)	✓		Flash-calcined kaolinite
Sutcu et al. (2014)	✓		Paper waste
Vegas et al. (2009)	✓		Calcined paper sludge
Wong et al. (2015)	✓		Wastepaper ash

with other waste streams. For this reason, the impact of the utilization of wastepaper in construction cannot be isolated from the bigger picture. Wastepaper, however, is associated with waste streams that are available in relatively smaller and isolated quantities, compared to the waste streams available in larger quantities such as from mining, general municipal waste, or agriculture or wastewater treatment. However, every little bit helps, and the less-abundant waste materials will continue to provide their own proportionate share of contribution to the bigger picture of attempting to mitigate the effect of global warming and climate change.

As resources become scarcer, the utilization of natural, industrial and agricultural waste, by-product and/or marginal materials, will undoubtedly increase. The initial impetus was from marginal materials that have been available in large volumes and in single localities. However, as these become depleted, the emphasis will slowly shift to a focus on efficient use of the remainder of the marginal materials, including the complex use of multiple materials that are in less abundance and in diverse regions.

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Sustainability of using waste rubber in concrete

23

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23.1 Introduction

Owing to the large increase in the number of cars worldwide, the accumulation of huge volumes of scrap tires has become a major waste management problem (Najim and Hall, 2010). A large amount of used rubber tires accumulate in the world each year (Fig. 23.1). For example, the waste tires in the United States are around 275 million (Papakonstantinou and Tobolski, 2006) and in the European Union there are about 180 million tires (Issa and Salem, 2013). Traditionally, tires have been stockpiled or illegally dumped or landfilled. However, none of them is a long-term solution. Accidental fires caused in stockpiled sites can rage for months releasing toxic fumes that are detrimental to the environment. Once a large pile catches fire, it is very hard, if not impossible, to extinguish. In some instances, large tire piles have been burning for several months with the fumes being visible for many miles. Due to the intense heat and smoke generation, firefighters have a hard time extinguishing these fires with conventional means. Also, air and soil pollution are even worse if attempts are made to extinguish the fire with foam or water. For this reason, scrap tire fires are often allowed to burn out in a more or less controlled manner until the entire pile is exhausted.

In general, the composition of tires is natural and synthetic rubbers, carbon black, metal, textile, and additives. These are potentially materials that can be recovered and reused to produce other materials and energy with low pollution risk compared with other alternative materials. The waste tires also can be used as alternative fuel in many energy-intensive industries, such as cement kilns, pulp and paper mills, steel mills, thermal power stations, and so on (Nehdi and Khan, 2001). During the last two decades, international research interest in the properties and potential uses for plain rubberized concrete has steadily grown. By using recycled tire rubber as concrete replacement material, the most beneficial potential for the use of industrial by-product is the environmental values. This effort will not only benefit the government in reducing landfill sites but also increase the economy growth in various sectors, especially the construction industry (Noor, 2014).

The basic building materials in concrete construction are primarily aggregate and cement. The adequate use of recycled materials can result in reduced cost potentials and may enhance performance; however, not all recycled materials are well suited for concrete-construction applications. The two main reasons for not utilizing a reclaimed material are the possible detriment to performance, and excessive cost (Issa and Salem, 2013). Tire-derived products are sometimes used to replace conventional construction

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Fig. 23.1 A view of waste tires.

material such as road fill, gravel, crushed rock, or sand. The benefits of using tire chips instead of conventional construction materials are, amongst others, reduced density, improved drainage properties, and better thermal insulation. The scrap tire chips have been successfully used in civil engineering applications, such as

lightweight fill for embankments and retaining walls,
leachate drainage material at municipal solid-waste landfills,
alternative daily cover at municipal solid-waste landfills,
insulating layer beneath roads and behind retaining walls.

Civil engineering applications of scrap tires are expected to become more widespread as more and more applications can be proven to be technically and economically viable. Utilization of waste tire rubber in construction technology has been widely recognized in road and pavement engineering. However, the use of rubber aggregate (from scrap vehicle tires) in concrete has become increasingly popular generating significant research interest over the past 20 years. Because of the large differences between Young's moduli of rubber and cement matrix, major differences in the mechanical properties are to be expected between concrete with conventional natural aggregate and rubber-containing concrete. Most significant is the loss in compressive and tensile strength as well as stiffness with increasing rubber content. The strength loss, which can be as high as 80% (Eldin and Senouci, 1993), is to be expected, since the rubber particles not only constitute weak inclusions, but also are responsible for significant tensile stresses in the cement matrix, which leads to cracking and failure. On the other hand, the rubber particles have a restraining effect on crack propagation, which leads to a significant increase in strain capacity, ductility, and energy absorption capacity (Topçu, 1997; El-Dieb et al., 2001).

23.2 Properties of rubber aggregates

Rubber tire aggregates are manufactured by processing nonreusable, worn vehicle tires into a range of classified particle-size distributions. The processing is mechanical and generally occurs either at ambient, cryogenic, or pyrolysis temperatures. Ambient temperature mechanical grinding is commonly used in industry where the scrap tire is cut into small pieces by using cracker mills and granulator techniques. In cryogenic processing, tire chips are cooled down and frozen in liquid nitrogen at a temperature of below −80°C. Below this glass transition temperature, rubber becomes nearly as brittle and glass and size reduction can be accomplished by crushing and breaking by using automatic hammers. In both processes, the steel wires in the tires are separated by applying a magnetic field and the rubber particles are separated from the wire mesh using vibrating sieves. Pyrolysis method refers to the thermal decomposition of scrap tires either in the absence or lack of oxygen. The process aims to recover the original raw materials from which the tire was made. These include carbon black, scrap steel, oil, and hydrocarbon gases. While the technical viability of the process has been proved, very few companies are using it.

Pretreatments may vary from washing rubber particles with water to acid etching, plasma pretreatment, and various coupling agents (Naik and Singh, 1995). In acid pretreatment, rubber particles are soaked in an alkaline solution (NaOH) for 5 min and then rinsed with water. This treatment enhances the strength of concrete containing rubber particles through a microscopic (a very small) increase in the surface texture of the rubber particles. Among the surface treatments tested to enhance the hydrophilicity of the rubber surface, a sodium hydroxide (NaOH) solution gave the best result. The NaOH treatment enhances the adhesion of tire-rubber particles to cement paste, and mechanical properties, such as flexural strength and fracture energy, were improved with the use of tire-rubber particles as addition instead of substitution for aggregate. The reduction in the compressive strength (33%) was observed, which is lower than that reported in the literature.

23.2.1 Classification of rubber aggregates

23.2.1.1 Shredded (chipped) rubber aggregate

This size of waste rubber represents the coarse aggregate (CA) equivalent and has been used as a partial replacement of gravel or crushed limestone (Ganjian et al., 2009). Tire shreds or chips involve primary, secondary, or both shredding operations. Production of tire chips, normally sized from 76 to 13 mm (Fig. 23.2A), requires both primary and secondary shredding to achieve adequate volume (quantity) reduction (Read et al., 1991).

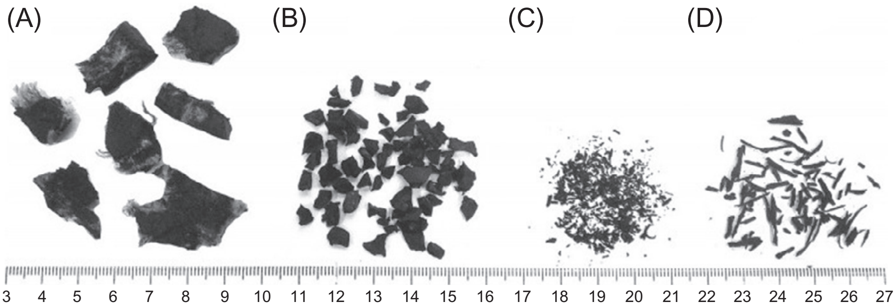

Fig. 23.2 Rubber aggregates: (A) shredded, (B) crumb, (C) granular, and (D) fiber.

23.2.1.2 Crumb-rubber aggregate

Generally, the following methods are used to convert scrap tires into crumb rubber. These methods are: (i) cracker mill process, (ii) granular process, and (iii) micromill process. The cracker mill process tears apart or reduces the size of tire rubber by passing the material between rotating corrugated steel drums. This process produces irregularly shaped torn particles having large surface area. The size of these particles varies from 5 to 0.5 mm and is commonly known as crumb rubber (Fig. 23.2B), which can be used as a partial replacement for fine aggregate (FA) such as natural sand.

23.2.1.3 Ground tire rubber

Ground rubber for commercial applications may be nominally sized as large as 19 mm to as small as 0.15 mm (Fig. 23.2C). It depends upon the type of size reduction equipment and intended applications. The processed used tires in ground rubber applications are typically subjected to two stages of magnetic separation and to screening. Various size fractions of rubber are recovered (Heitzman, 1992).

23.2.1.4 Slit (fiber) rubber aggregate

Fiber rubber wastes are produced in tire-cutting machines. These machines can slit the tire into two halves or separate the sidewalls from the thread of the tires. A handful of researchers has used shredded rubber in the form of a short fiber, typically between 8.5 and 21.5 mm in length with an average of 12.5 mm (Baoshan et al., 2004), and in the form of strips 68 mm long (Emiroglu et al., 2007) (Fig. 23.2D).

23.3 Fresh rubberized concrete properties

23.3.1 Slump

A significant reduction is obtained in slump of concrete as the rubber aggregate content increases. This reduction is probably due to the higher level of interparticle friction that occurs between the rubber aggregate and the other mix constituents as well as the overall reduction in the unit weight of the plastic mix (Topçu and Bilir, 2009). The slump also decreases with increase of rubber content by total aggregate volume (Khatib and Bayomy, 1999). The mixtures made with fine crumb rubber are more workable than those with coarse tire chips or a combination of tire chips and crumb rubber (Fig. 23.3). At rubber contents of 40% by total aggregate volume, the slump is near zero and the concrete is not workable by hand. Such mixtures have to be compacted using a mechanical vibrator. Mixtures containing fine crumb rubber are, however, more workable than mixtures containing either coarse tire chips or a combination of crumb rubber and tire chips. When the rubber types are considered, despite the decrease in measured slump, observation during mixing and casting showed that increasing the crumb content in the mix still produced a workable mix (as indicated above) in comparison with the control mix (Batayneh et al., 2008). The concretes incorporating shredded rubber achieved workability similar or better than a control concrete without rubber particles

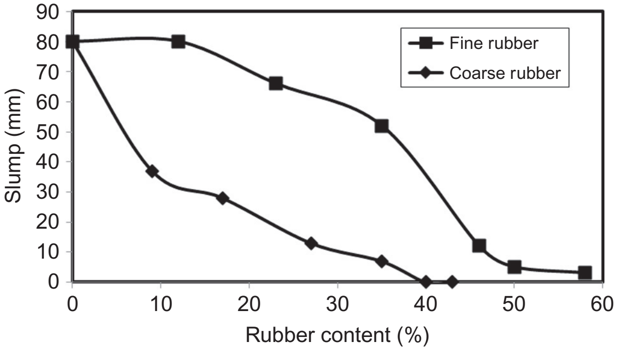

Fig. 23.3 Change in slump of concrete with addition of rubber (Nehdi and Khan, 2001).

(Raghvan et al., 1998). Increasing the rubber content up to 38 wt% at the same water to cement ratio will reduce the mix flowability but greatly assist the control of bleeding and segregation (Pierce and Blackwell, 2003).

23.3.2 Air content

Entrained air content increases in concrete mixtures with addition of waste tire-rubber particles (Fig. 23.4). Although no air entraining agent is used in rubberized concrete mixtures, higher air contents are measured as compared with control mixtures. This may be due to the nonpolar nature of rubber particles and their tendency to entrap air in their rough surfaces. Also, when rubber is added to a concrete mixture, it may attract air as it has the tendency to repel water, and then air may adhere to the rubber particles. Therefore, an increase in the rubber content results in higher air content in rubberized mixtures, thereby decreasing the unit weight of the mixtures.

23.3.3 Density

Density, also known as unit weight, is very important as a part of the design process of concrete because unit weight of concrete will classify the categories of concrete.

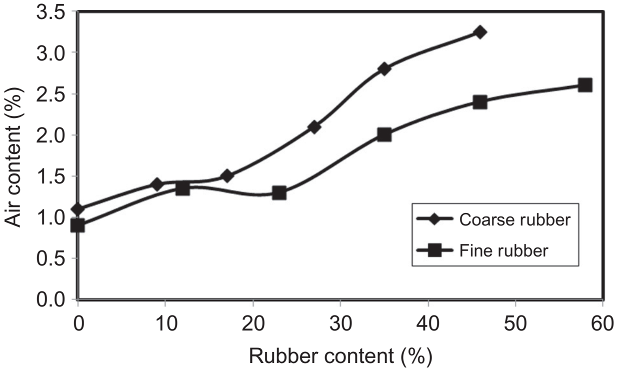

Fig. 23.4 Air content of rubberized concrete (Nehdi and Khan, 2001).

Factors affecting concrete density include aggregate density, air content, and the water and cement content in the design. The increase of the rubber content in the concrete mix results in a decrease in the unit weight of the mixtures. Because of low specific gravity of rubber particles, where δ is about 0.53, unit weight of mixtures containing rubber decreases with the increase in the percentage of rubber content. The decrease in unit weight of rubberized concrete is negligible when rubber content is lower than 10–20% of the total aggregate volume (Khatib and Bayomy, 1999). The density of rubberized concrete also decreases with increased maximum particle diameter. With 50% fine rubber and coarse rubber replacement, for example, up to 75% reduction in dry density has been achieved compared with a range of 10–30% dry density reduction for FA replacement only (Fattuhi and Clark, 1996). The increase of air entraining of concrete mixtures with rubber-waste additive can be explained by higher porosity of the rubber-waste particles than sand particles and very porous surface of rubber particles (Skripkıūnas et al., 2007; Batayneh et al., 2008). Vibration time has an effect on the density of rubberized concretes. The fresh rubberized concrete that was vibrated for 30 s showed distinct segregation. Therefore, vibrating rubberized concretes more than 15 s is proposed. The lower vibration time may also cause lower fresh-state density than is found for plain concrete (Bravo and Brito, 2012).

23.4 Mechanical properties of rubberized concrete

23.4.1 Compressive strength

The size, proportion, and surface texture of rubber particles noticeably affect compressive strength of rubberized concrete mixtures. The concrete mixtures with tire chips and crumb-rubber aggregates exhibit lower compressive strengths than plain concrete. There is about 60% reduction in compressive strength when CA was replaced in ratio of 45% by coarse crumb-rubber chips. However, a reduction of about 40% in compressive strength is obtained when FA is replaced with 45% by fine crumb rubber (Fig. 23.5). Both of these mixtures demonstrate a ductile failure and the ability to absorb a large amount of energy under compressive and tensile loads. Khatib and Bayomy (1999) and Topçu (1995) also showed that the addition of coarse rubber chips in concrete lowered the compressive strength more than the addition of fine crumb rubber. However, results reported by Fattuhi and Clark (1996) indicate the opposite trend. Studies have indicated that if the rubber particles have rougher surfaces or were given a pretreatment, then better and improved bonding may develop with the surrounding matrix, and, therefore, that may result in higher compressive strength. The replacement of aggregates with granulated rubber waste deteriorates mechanical properties of concrete (Eldin and Senouci, 1993; Topçu and Avcular, 1997a; Lee et al., 1993). The decrease of compressive strength of concrete after modification with rubber waste is due to the more elastic and softer rubber particles when compared with the sand particles (Khaloo et al., 2008). The second reason for concrete compressive-strength reduction is significantly the lower compressive strength of the rubber particles compared with the strength of concrete aggregates (Topçu, 1995; Taha

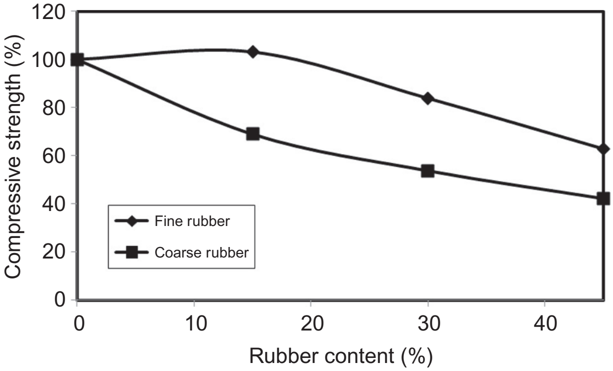

Fig. 23.5 Compressive strength of rubberized concrete (Topçu, 1995).

et al., 2008). Deterioration of the mechanical properties of concrete with rubber additives is also explained by low adhesion among the rubber particles and cement matrix (Issa and Salem, 2013).

The better the bonding they develop with the surrounding matrix obtained by roughening the rubber particles used in concrete, the higher the compressive strength achieved. Pretreatments vary from merely washing the rubber particles with water to acid etching, plasma pretreatment, and various coupling agents. This increases the strength of rubberized concrete through a microscopic increase in surface roughness of the rubber, which improves its attachment to the cement paste (Fig. 23.6). In general, concrete containing washed-rubber particles shows about 16% higher compressive strength than concrete containing untreated rubber aggregates (Rostami et al., 1993).

The micrograph of Fig. 23.6 contrasts the poor bonding of cement paste with rubber and its tight bond with natural sand. It is an additional factor contributing to the

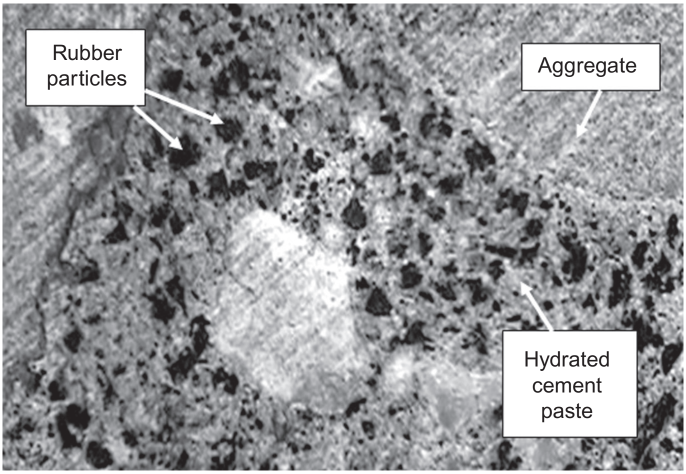

Fig. 23.6 Microscopic view of crumb-rubber distribution (Kaloush et al., 2005).

decrease in the compressive strength of the composite material. Appropriate rubber aggregate coating enhanced the rubber/cement-paste bond and succeeded in improving the compressive strength of rubberized composites (Taha et al., 2008). This finding will be taken into account in further studies in order to establish the impact of the rubber/cement-paste bond on the physical properties and durability of the material. On the one hand, rubber aggregates could be considered as weak zones, and when a microcrack runs into their interface with the cement paste, stress concentration around its tip is reduced (Karakurt, 2014).When compared to air pores, they have a higher capacity of energy absorption, tending to eliminate the mechanism for further propagation. Due to the elastic nature of rubber, failure of the rubberized concrete is elastic, that is, it absorbs high energy before failure. Rubberized concrete is tough compared with concrete matrix without rubber. After loading the samples, cracks start first at the softest areas of the specimens. Increasing rubber content increases the number of cracks and their width, which leads to collapse of the concrete specimens.

23.4.2 Stress–strain behavior

Strains of the concrete with tire rubber during the initial stages of loading are higher than concrete without rubber waste. It can be explained by the great deformability of rubber-waste particles under the low loads. Most of these deformations of rubber-waste particles have plastic nature. Therefore, concrete with tire-rubber waste exhibits higher deformations than nonrubberized concrete (Liu et al., 2013). The specimens withstood measurable postfailure compression loads and underwent significant displacement without full disintegration. Displacements and deformations are partially recoverable upon loading. Rubber particles having low modulus of elasticity produce high internal tensile stress that is perpendicular to the direction of the compressive load applied (Fig. 23.7). Cement paste shows early failure because of its weakness against tension.

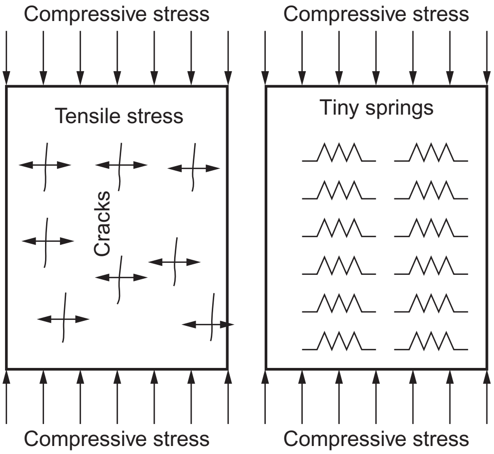

Fig. 23.7 Schematic representation of rubberized concrete under compression (Nehdi and Khan, 2001).

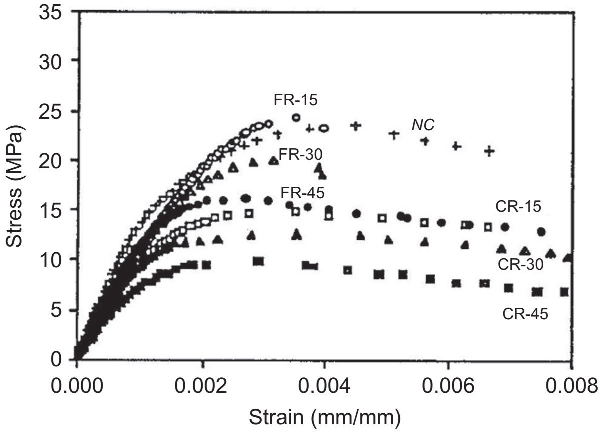

Fig. 23.8 Stress–strain of rubberized concrete under compression load (Topçu, 1995).

Rubber chips behaving like springs delay the widening of the existing cracks (Topçu, 1997; Chung and Hong, 1999). Continuous application of compressive load generates more cracks and widens the already present ones. When the bond between cement paste and rubber is overcome, failure occurs.

When the stress–strain diagram of rubberized concrete is analyzed (Fig. 23.8), we see that plain concrete reaches the ultimate strain around 0.2%. The concrete having 15% rubber particles shows behavior similar to normal concrete. With a mixture of concrete and rubber particles, maximum stress points fall while the strain increases at the failure point in rubberized concrete strain values change between 0.3% and 0.5% against maximum stress.

23.4.3 Modulus of elasticity

The use of rubber as addition material in concrete reduces the static modulus of elasticity because of rubber particles is effective in increasing deformability of concrete. Using rubber waste in concrete reduces the modulus of elasticity (Zheng et al., 2008). The modulus of elasticity is related to concrete compressive strength, and the elastic properties of aggregates. The larger the amount of rubber additives added to concrete, the smaller the modulus of elasticity obtained. By using rubber waste in concrete, the dynamic modulus of elasticity also decreases compared without rubber additives. The reduced static and dynamic modulus of elasticity in concrete with waste rubber aggregates may be explained by the low modulus of elasticity of small rubber particles, which is much lower than the aggregate modulus of elasticity (Topçu and Avcular, 1997b).

23.4.4 Toughness

Toughness is defined as energy absorption capacity and is generally defined as the area under load–deflection curve of a flexural or stress–strain curve of a compressive test of specimen. Toughness of rubberized concrete is higher than that of the control

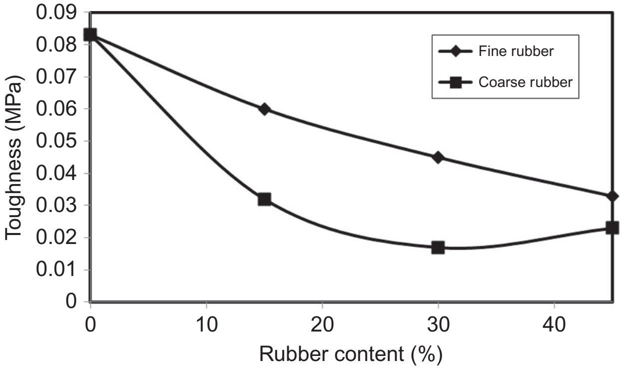

Fig. 23.9 Toughness of concrete depending on rubber content (Topçu, 1995).

concrete mixture. However, the toughness of rubberized concrete with 10% rubber is lower than that of rubberized concrete with 5% rubber because of the decrease in compressive strength. Changes in toughness values with addition of rubber are determined by measuring the areas under the stress–strain diagrams. Although decreases in toughness with addition of rubber are seen (Fig. 23.9), some changes are witnessed in the energy capacities consumed during the fracture. Since the rubberized concretes absorb more energy, they can show more strain at the time of fracture (Topçu, 1997).

Toughness values in terms of energy capacities consumed at the time of fracture are investigated in two different ways as plastic and elastic properties. Plastic energy is defined as energy consumed during the failure and never recovered again. Elastic energy is known as the recovered deformation energy to be obtained just before fracture (Topçu, 1995). The originally high elastic energy capacities of normal concretes begin to decrease with the addition of rubber, and originally low plastic energy capacities begin to increase (Fig. 23.10). Concrete with high plastic energy shows higher deformation at the time of fracture and could absorb more energy. With rubber mixed into the concrete, plastic energy capacities can be increased (Topçu, 1997).

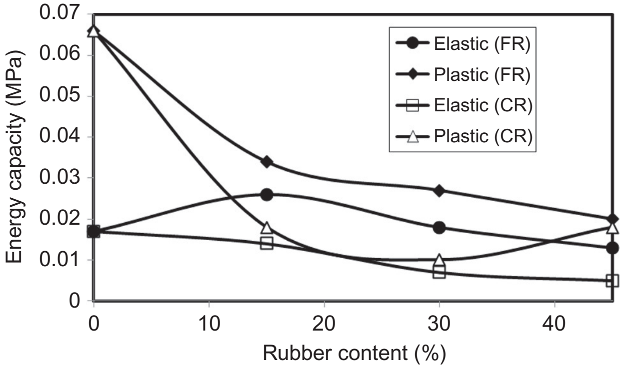

Fig. 23.10 Energy absorption of concrete depending on rubber content (FR, fine rubber; CR, coarse rubber) (Topçu, 1995).

23.4.5 Impact resistance

Many concrete structures are often subjected to short duration dynamic loading. These loads come from sources such as impact from missiles and projectiles, wind gusts, earthquakes, and machine vibrations. Impact is a complex dynamic phenomenon involving crushing shear failure and tensile fracturing. The use of waste tire rubbers is found to be advantageous in both static and impact conditions. The impact resistance of concrete is increased when rubber aggregates are added to the mixture. It is argued that this increased resistance is derived from an increased ability of the material to absorb energy. Furthermore, the increase in these properties became more prominent in the concrete samples with larger rubber aggregate. These altered characteristics are attributed to the fiber nature of rubber aggregate, which gives the concrete its flexibility and capacity to take in strokes. For determining the impact resistance of rubberized concrete, ACI Committee 544 (2005) proposed a drop-weight impact test to demonstrate relative brittleness and to quantify the impact resistance of concrete. The test is widely used because it is simple and economical.

The results obtained from this test are often noticeably scattered. Sallam et al. (2008) studied the effect of ground-waste tire rubber addition on the impact resistance of normal strength concrete. Repeated drop-weight impact tests were carried out on discs of 150 mm diameter and 63 mm height. Test results showed that the presence of crumb rubber of small size in concrete increased its resistance to crack initiation under impact load. The rubber of small size has no particle bridging effect, hence, the mode of failure of rubberized concrete under static and impact compression was the same as that of plain concrete. The impact test failed to differentiate between the impact resistance of rubberized concretes with different rubber contents (Fig. 23.11).

23.4.6 Splitting strength

The strain at failure increases as the rubber content is increased, but the tensile strength decreases. Higher tensile strain at failure is indicative of more ductile mixes and more energy absorbent mixes. In fact, the product of the tensile strength and strain at failure is

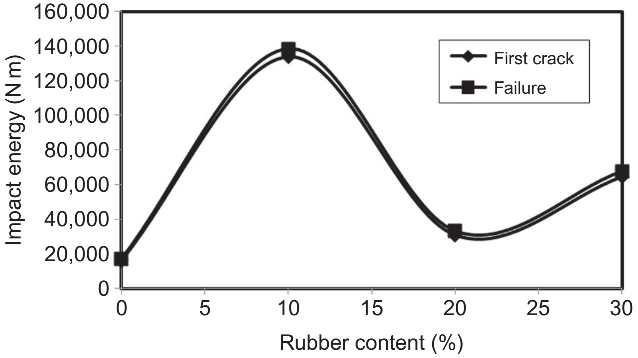

Fig. 23.11 Impact resistance of rubberized concrete (Sallam et al., 2008).

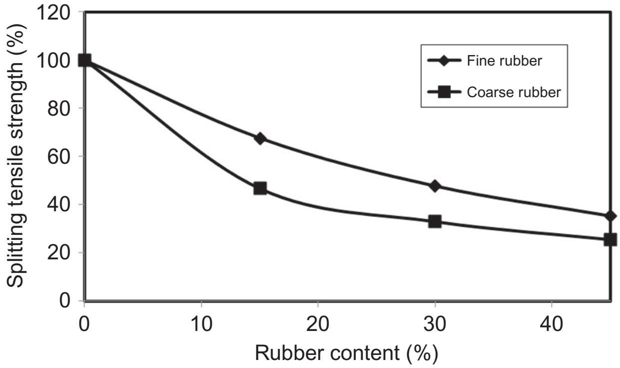

Fig. 23.12 Splitting tensile strength of C20 rubberized concrete (Topçu, 1995).

indicative of the energy absorbed by each mix until failure. In splitting tensile strength, specimens show high capacity of absorbing plastic energy. The failed specimens withstood measurable postfailure loads and underwent significant displacement, which is partially recoverable. Thus, the concrete mass was able to withstand loads even when it was highly cracked (Liu et al., 2013). This may be due to the ability of rubber aggregate to undergo large elastic deformation before failure. Specimens containing rubber did not exhibit brittle failure under compression due to rubber's plastic behavior. Splitting is gradual depending on the type and amount of rubber used (Topçu, 1995). Fig. 23.12 shows the splitting tensile strength of a C20 concrete with respect to the amount of tire chips added. It is clearly seen that the splitting strength of rubberized concrete decreased by addition of rubber particles. About 65% decrease in splitting strength of concrete is observed for 45% addition of fine rubber particles compared with decrease of about 75% for the same content of coarse rubber addition. This suggests that the coarse tire aggregate lowered the splitting strengths more than the fine tire chips. This is because of weak interfacial bonds between the cement paste and tire aggregate.

23.4.7 Flexural strength

Using rubber aggregate reduces the flexural strength of concretes. Even though rubber particles decrease the flexural strength of concrete; rubberized concrete (RC) specimens do not collapse suddenly under bending load during the flexural test (Fig. 23.13) due to high deformations of rubber particles (Liu et al., 2013). Besides, the flexural strength of concretes at 28 ages do not decrease very much until rubber content reaches 30% in low water to cement ratios, specifically, in 0.40 and 0.43 (Fig. 23.14). This is due to those rubber particles that are distributed in the mixes preventing crack development similar to behavior of steel fibers. Moreover, the brittleness index decreased with rubber additions after 15%, and it shows that there is a transition from brittle ductile behavior (Topçu, 1997).

23.4.8 Abrasion strength

Abrasion is described as the behavior of the construction element against destructive influences due to scraping, rubbing, skidding, or sliding of the object on the surface,

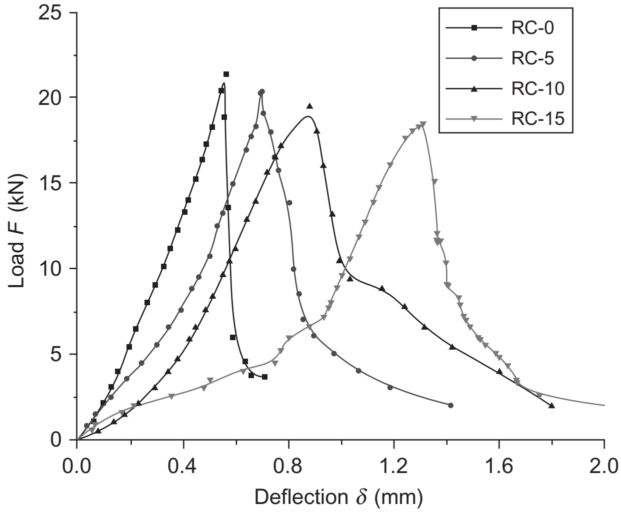

Fig. 23.13 Load-deflection of rubberized concrete (RC) under three point flexural test (Liu et al., 2013).

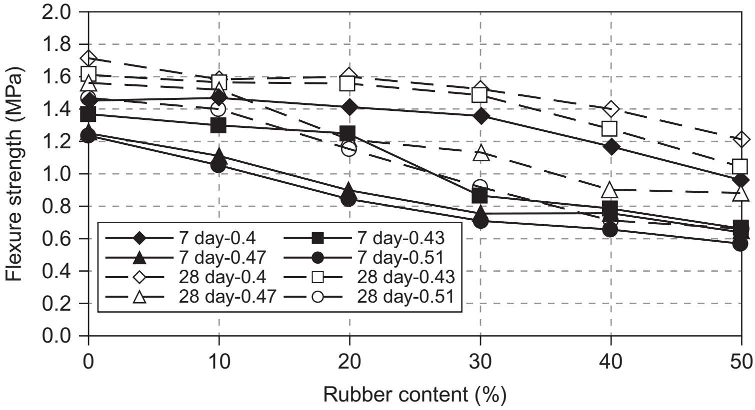

Fig. 23.14 Flexure strength of rubberized mortar at 7 and 28 days of age (Uygunoğlu and Topçu, 2010).

which leads to surface erosion. Abrasion resistance is mainly a consideration on the structure exposed to heavy environment such as sea waves. The rate of wearing loss depends on the type and size of abrasion material and also on loading pressure toward the concrete surface. Wearing can lead to spalling of the concrete surface, thus causing exposure of embedded steel reinforcing, which can lead to corrosion.

The addition of crumb rubber reduces the compressive strength but increases notably the abrasion resistance of the concrete (Kang et al., 2012). Results of the effect of crumb rubber replacement at 10%, 15%, and 20% on surface abrasion wear resistance are shown in Fig. 23.15. At 10% there is noticeable improvement in abrasion

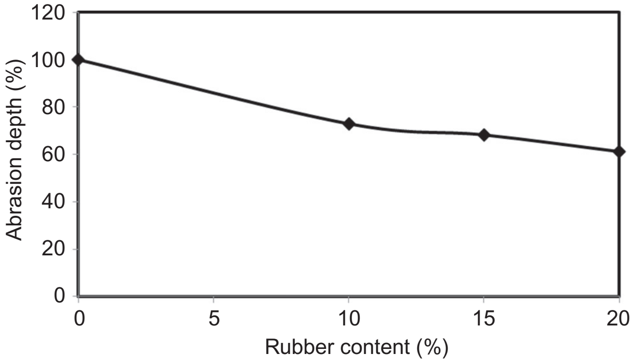

Fig. 23.15 Abrasion strength of rubberized concrete (Noor, 2014).

resistance compared to the control, and the resistance continues to increase at higher rubber content. The abrasion depth was 73%, 68%, and 61% for concrete with 10%, 20%, and 30% rubber addition, respectively, compared with 100% for the control. It is clearly seen that crumb rubber addition as sand replacement to concrete provides good resistance than that control mix. This improvement may be due to the restriction of surface grinding due to the crumb-rubber position which was beyond the smooth surface and acted like a brush. Also, rubber can resist against wear better than cement matrix, thus during the test, part of rubber particles out from the surface and prevent the cement matrix from the wearing action (Rangaraju and Gadkar, 2012; Thomas et al., 2014).

23.4.9 Bond strength of rubberized concretes

Increasing rubber content decreases the bond strength of concrete composite. Bond strength is usually measured in terms of load required to pull out the steel bar from the concrete samples. There is approximately 14%, 45%, 55%, and 61% decrease in the bond strength with the addition of 0%, 10%, 20%, 30%, and 40% of rubber content, respectively, in the rubberized specimens at 28 days (Fig. 23.16). Moreover, as the

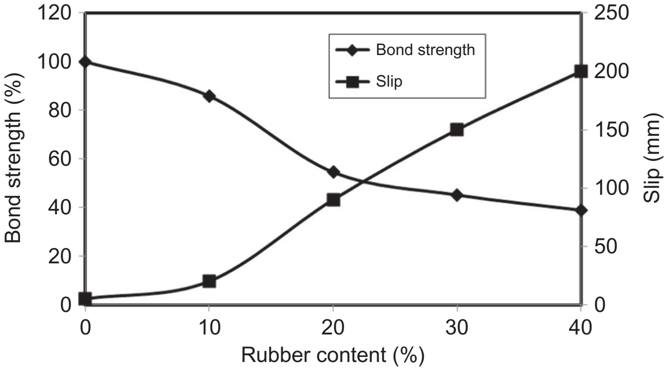

Fig. 23.16 Bond strength of rubberized concrete depending on rubber content (Bala et al., 2014).

rubber content is increased, slip of bar from the concrete during the pullout is also increased. The weakness of the bond between scrap rubber and cement matrix can be seen by the ease with which rubber aggregates can be removed from the crushed sample by simply using one's fingers (Bala et al., 2014).

23.5 Physical properties of rubberized concrete

23.5.1 Water absorption and permeability

Replacement of rubber causes increase in water permeability depth and water absorption of concrete for CA replacement. Increase in water absorption with addition of rubber particles is essentially because rubber particles need higher water content to maintain a similar workability in all mixes and also because of the greater void volume between rubber particles and the cement paste. There is an increase of around 15% with a replacement of 15% of normal aggregate by rubber particles (Bravo and Brito, 2012). Also, high rubber replacement shows a very high permeability in concrete. This is due to the weak bond between rubber and concrete material. In general, the increase in permeability is attributed to increase in open porosity with increase in rubber particles. The nonconformity of gradation of rubber particles and sand may also be a reason. Permeability of concrete is increased by the interconnection of the flow path in between cracks of concrete. This leads to progression of aggressive chemicals and water, which causes deterioration of concrete. Permeability of concrete not only depends on the crack width but also on crack length and number (Benazzouk et al., 2007).

23.5.2 Apparent porosity and density

As expected, the unit weight, or dry density, of concrete decreases with increasing content of rubber particles due to its low unit weight (ie, density or specific gravity). The density of rubberized concrete decreases both with increased replacement ratio and with increased maximum particle diameter. Smaller rubber aggregates result in lower density and apparent porosity as compared to larger rubber aggregates. Rubber particles that have longitudinal shape and particle size of more than 4 mm cause porosities in the concrete. Also, it has been suggested that using ultrafine materials in the rubberized concrete production provides an increase in homogeneity and a decrease in the number of large pores in cement paste. Higher rubber content causes reduction in density while lower water content results in higher apparent density. This is likely due to the entrainment of air by the rubber aggregates at the particle–paste and particle–particle interfaces, and the increase in the porosity of the paste with decreasing powder content (Siddique and Naik, 2004).

23.5.3 Capillary water absorption

Capillary absorption represents the main mechanism for water and water vapor transport in concrete. The capillary absorption of concrete increases with an increasing content of rubber particles due to its poor bonding with hydrated cement paste. The size of the replaced aggregate is important to the increase in water absorption by capillarity since all the coarse rubber particles have higher absorption rates than the fine rubbers. When the rubber surface is considered in capillary water absorption, rubberized concrete with mechanically ground fine rubber has lower water absorption by capillarity than rubberized concrete made with fine cryogenic rubber particles. This is due to the better adherence between the cement paste and the mechanically ground rubber particles, due to their greater roughness (Bravo and Brito, 2012).

23.5.4 Drying shrinkage

Shrinkage phenomenon is proportional to the volume of cement paste. The volume of paste has a noticeable effect on shrinkage. The expected results may be (i) shrinkage increases with increasing water to cement ratio (w/c) due to the increase in the nano/meso porosity (water content) of the paste and (ii) in the case of increasing rubber content (this may be complicated due to many factors) (Uygunoğlu and Topçu, 2010). Specifically, the rubber may change moisture diffusion through the material and the restraining effect of the rubber particles. However, given that the stiffness of the rubber is much less than the paste (and the sand aggregate), the rubber may not decrease shrinkage due to restraint, rather an increase in shrinkage may be noted (as noted for w/c > 0.43). As a first thought, it is expected this should be consistent for all the materials evaluated with increasing rubber content.

Drying shrinkage increased significantly in rubberized mortars with high rubber content (40% and 50%) and in water to binder (w/b) ratio of 0.51 at 28 ages (Fig. 23.17). This is due to the increase of porous structure in rubberized mortars with high water content. An interesting feature of this study is drying shrinkage of rubberized mortars with up to 30% scrap-rubber content as the w/b ratio of rubberized mortars increases. Drying shrinkage of rubberized mortars with 10%, 20%, and 30% scrap-rubber content is lower than that of control specimens. It is well known from the literature that drying shrinkage of concrete increases with increase of porosity. However, the addition of scrap rubber had a significant effect

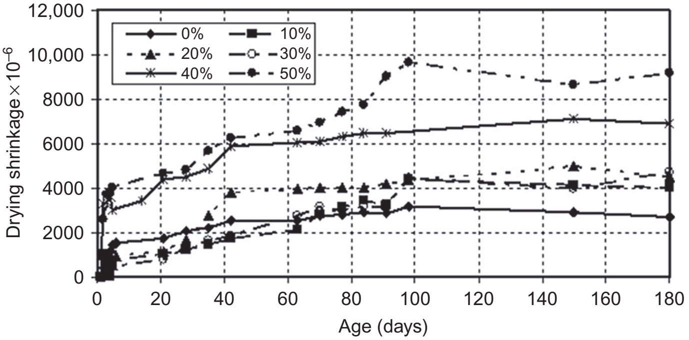

Fig. 23.17 Drying shrinkage of rubberized mortars (water:binder is 0.51; Uygunoğlu and Topçu, 2010).

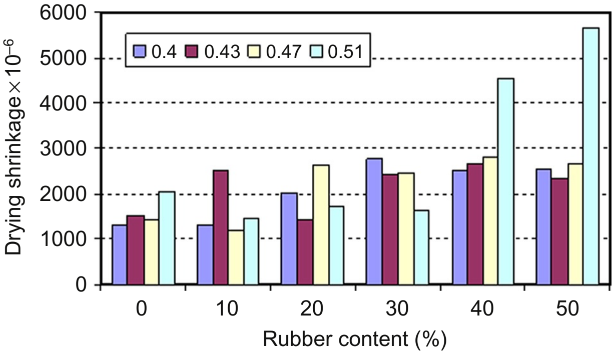

Fig. 23.18 Drying shrinkage of rubberized mortars at 28 days depending on water to binder ratio (Uygunoğlu and Topçu, 2010).

in reduction of drying shrinkage of rubberized mortars during the first month (Uygunoğlu and Topçu, 2010). On the other hand, the maximum shrinkage was usually obtained for those rubberized mortars produced in the highest w/b ratios at 28 ages (Fig. 23.18).

The influence of scrap rubber in reducing drying shrinkage is very significant in lower w/b ratios. The addition of scrap rubber, specifically from 10% to 50%, had a significant effect in increasing drying shrinkage of rubberized mortars that produced with w/b ratio of 0.51 after 28 days. This is due to the increase of porous structure in rubberized mortars with high w/b ratio and scrap-rubber content. Furthermore, in rubberized mortars with high w/b ratios, free water evaporates, thus, porosity increases after 6 months. Consequently, the highest drying shrinkage values were observed in rubberized mortars with the high scrap-rubber content (40% and 50%) and with the highest w/b ratio (Uygunoğlu and Topçu, 2010).

23.5.5 Coefficient of thermal expansion

Thermal expansion is an important factor in all types of concrete structure including bridges, roads, railways, and buildings. Restrained thermal stresses have been shown to significantly increase the risk of explosive spalling of high-performance concrete exposed to fire. The differential expansion that occurs between cement paste and aggregate will give rise to high internal stresses, which may be critical in the case of large temperature changes (Uygunoğlu and Topçu, 2009). The addition of crumb rubber (27 kg/m^3) reduced the coefficient of thermal expansion (CTE) values by about 29% when compared to plain concrete. These results indicate that the crumb mixes are more resistant to thermal changes. Fig. 23.19 compares the CTE values of rubberized concretes with other commonly used mixtures, such as conventional, fiber-reinforced, and fly-ash added concrete. Comparing the average CTE results for all the four mixes in Fig. 23.19, rubberized concrete with 136 kg/m^3 waste tire content has the lowest value. It was 50% less than the value of the plain concrete (Kaloush et al., 2005).

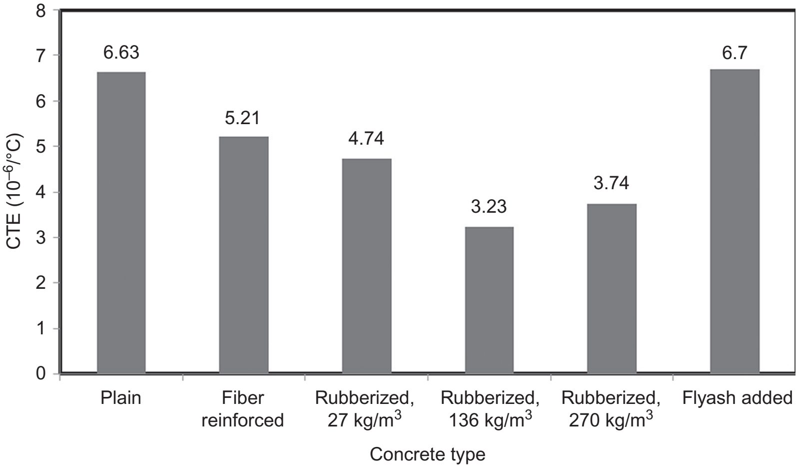

Fig. 23.19 Comparison of coefficient of thermal expansion (CTE) values for different types of concretes (Kaloush et al., 2005).

23.6 Durability properties of rubberized concretes

Even if the strength of rubberized concrete is significantly reduced as compared with the control concrete, certain durability properties are shown to be improved by many studies conducted by other researchers.

23.6.1 Freeze–thaw resistance

As the water in moist concrete freezes, it produces pressure in the pores of the concrete because when water freezes, it expands about 9%. If the pressure developed exceeds the tensile strength of the concrete, the cavity will dilate and rupture. The accumulative effect of successive freeze–thaw (F–T) cycles and disruption of paste and aggregate can eventually cause expansion and cracking, scaling, and crumbling of the concrete (Fedroff et al., 1996). Freeze–thaw damages have continued to be a persistent problem in the durability of concrete structures especially in cold regions. In spite of strength reduction in rubberized concrete, there is less freeze–thaw damage compared to the control concrete test (Topçu and Demir, 2007). Recycled particles of sustainable rubber crumb have a low modulus of elasticity (Khaloo et al., 2008). Due to this, it is postulated that its inclusion as a constituent of concrete production introduces pressure-relief chambers. These chambers alleviate hydrostatic pressure via a mechanism akin to that afforded by air entrainment, thus promoting freeze–thaw protection. If the rubber crumb does entrain air, it provides further concrete protection. The investigations on various parameters such as volume of open and closed air pores, size and distribution of air pores, and freezing and thawing factor showed that addition of rubber to the cement matrix increased the porosity of the matrix and air entrainment was higher for smaller rubber particle size. The rate of compressive–strength decrease is lower compared to control for specimens exposed to freeze–thaw cycles (Fig. 23.20).

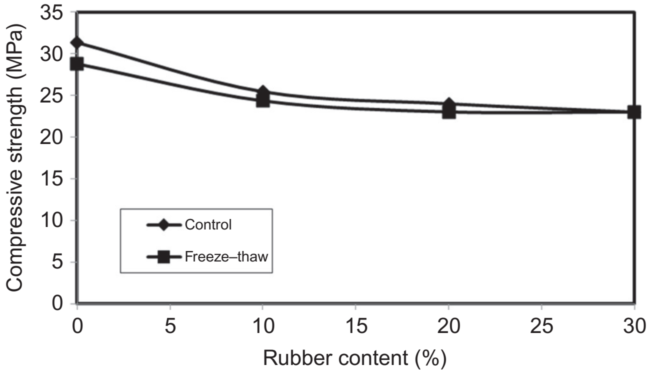

Fig. 23.20 Change of compressive strength of rubberized concrete under freeze–thaw (Khaloo et al., 2008).

The changes in dynamic elasticity modulus values are also similar to the changes in compressive strength values.

Air entraining agents are widely used to resolve this durability issue. However, lots of factors impact the effectiveness of these admixtures (including aggregate properties, their gradation, amount of cement paste, etc.) and often bring difficulty in deciding the right amount of dosages to obtain a stable air void system in fresh and hardened state of concrete. Rubberized concrete can be used to produce frost-resistance concrete without air entraining agents by controlling the parameters of cement and rubber since the rubberized concrete increases the closed porosity (Rangaraju and Gadkar, 2012). Investigations on the mechanical properties of rubber cement composite aerated by artificially entrapping air voids by using new air entraining agents indicated improvement in workability and air entrainment with higher stability of air bubbles in the matrix. The rubber-waste additives in rubberized concrete act as an absorber and balance all internal stresses in concrete paste caused by hydrostatic water pressure (Benazzouk et al., 2006). Inclusion of fine crumb-rubber aggregates as partial replacement to the sand incorporates a system of well-distributed air bubbles into the concrete matrix that, in turn, helps to improve freeze–thaw resistance. Finer size fractions shows very good performance in the F–T test at higher replacement levels of 16% and 24% by volume of sand. Inclusion of finer crumb-rubber aggregates increases the air bubbles in the concrete, which ultimately helps in improving F–T resistance of portland cement concrete (Richardson et al., 2011). Finer size crumb–rubber aggregates may be effectively used instead of air entraining admixtures to improve F–T resistance. The inclusion of rubber aggregates into the concrete reduces the strength significantly, but may be useful for improving durability properties such as freeze–thaw resistance.

23.6.2 Chloride ion permeability

The concrete used for road and bridge surfaces in cold areas is often eroded by chloride ions after ice-removing salt treatment. Furthermore, the concrete exposed to seawater will be eroded by chloride ions. Chloride ions move through the concrete to the level of the rebar. When this occurs in combination with wetting and drying cycles and in the presence of oxygen, conditions are right for reinforcement corrosion, which even damages the alkaline passivation film and eventually causes electrochemical reaction and section loss of reinforcing bar (Güneyisi et al., 2004). An increase in the chloride diffusion coefficient of the rubberized concrete occurs when the addition ratio of rubber particles is increased. Increasing the size of the rubber particles leads to a higher chloride diffusion coefficient. Also, fine cryogenic rubber particles have less resistance to chloride penetration than fine mechanically ground rubber aggregates because of the better adherence of cryogenic rubber particles to the cement paste (Bravo and Brito, 2012).

23.6.3 Carbonation depth

Carbonation depth of concrete increases with the replacement ratio of natural aggregate by waste tire rubbers. The increasing of carbonation depth may be due to the higher water content of rubber particles that needs to maintain the workability with plain concrete, void volume between rubber particle and the cement paste. The 28-day carbonation depth increased 56% when 15% of coarse natural aggregate was replaced. Increasing the size of the replaced aggregate led to a slight growth of the carbonation depth. Once again it is the same trend as in the water absorption tests (Bravo and Brito, 2012).

23.6.4 Fire performance of rubberized concrete

The compressive strength of the normal strength concrete under the effect of high temperature decreases with increasing rubber addition. There is an increase in the compressive strengths at 300°C (Fig. 23.21). It is thought that the reason for this is that hydration of the mortar is completed in this temperature (Topçu and Demir, 2007).

However, the specimen of high-strength concrete without rubber particles presents spalling on the exposed side, while the specimens with rubber do not show spalling when the volumetric fraction of rubber increased. When the rubber particles are

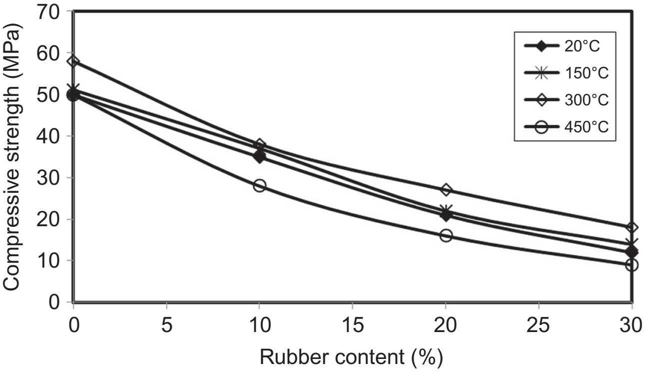

Fig. 23.21 High temperature effect on rubberized concrete (Topçu and Demir, 2007).

burnt, they allow water vapor to escape from the sample, reducing stress due to vapor pressure. Small holes can be observed in the fire-exposed surface of the specimens with rubber fiber. No holes are present on the specimen without rubber. High-strength concretes show a tendency of explosive spalling when subjected to rapid heating, as in the case of fire. This behavior is mainly due to the lower permeability that results in a buildup of pore pressure within the hydrated cement paste. Some solutions, as the inclusion of polypropylene fibers to allow water vapor to escape from the concrete matrix, have been proposed to reduce the risk of spalling. Use of rubber aggregates in the concrete may be an alternative path to a solution for spalling of high-strength concrete under fire or high temperature (Hernandez-Olivares and Barluenga, 2004).

23.6.5 Seawater effect and acid attack

Concrete in marine environments suffers deterioration, which may be due to the effects of chemical reaction of seawater constituents with cement hydration products, crystallization pressure of salts within concrete when one face of the structure is subject to wetting and others to drying conditions, corrosion of embedded steel in reinforced or prestressed members (Huynh and Raghavan, 1997; Olutoge and Amusan, 2014). Cement-based mortars containing the rubber particles are kept in NaCl solutions (salty water) representing the effect of seawater when some of the produced specimens are kept in normal curing conditions for 28 days. At the end of 28 days, the compressive strength of the mortar under the seawater effect decreased according to the mortar specimens under normal curing conditions. In the mortar specimens kept in salty water for 28 days, the compressive strength fell from 33.75 to 5.18 MPa (Fig. 23.22). In the mortar specimens with 10%, 20%, 30%, and 40% rubber additions, compressive-strength decreases compared with the normal mortar specimens, and these decreases are affected more by salty water (Topçu and Demir, 2007).

The rubberized concretes are more vulnerable in acidic medium when compared to seawater.

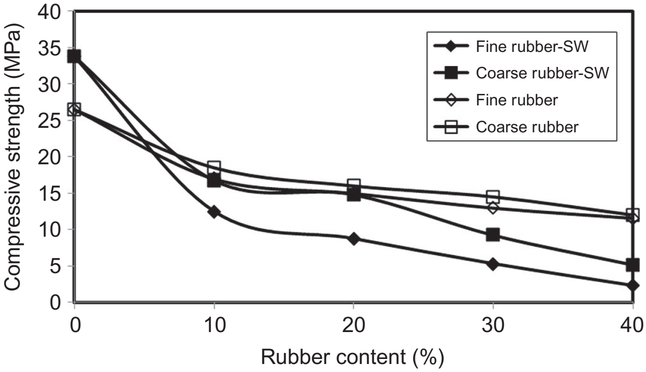

Fig. 23.22 Seawater (SW) effect on compressive strength of rubberized mortars (Topçu and Demir, 2007).

Rubberized concrete specimens having the same strength as plain concrete show negligible reduction in mass when immersed in seawater. This may be due to the replacement of FAs by rubber, which is less reactive in chloride environments (Ganesan et al., 2012).

23.7 Use of mineral additives in rubberized concrete

Modification of concrete properties by the addition of some mineral additive material is a popular method in concrete research. Rubberized concrete properties are also investigated in this manner (Elchalakani, 2015). In general, silica fume, fly ash, and metakaolin are used as supplementary cementitious materials in rubberized concrete. In order to increase concrete strength, using of silica fume in concrete has attracted much attention. The silica fume particles help filling the nanovoids in cement paste, and they provide a denser structure. This results in the increment of concrete strength and durability.

The increase of compressive strength in the presence of silica fume is due to its high pozzolanic activity. The silica fume particles react with the calcium hydroxide from cement hydration reaction and form new calcium silicate gels (C-S-H). The gels contribute to the strengthening of concrete (Sohrabi and Karbalaie, 2011). Silica fume is also effective in resistance to chloride penetration. Overall, silica fume improved the performance of rubberized concrete and increased the resistance to crack initiation (Güneyisi et al., 2004). However, silica fume addition has no considerable effect on compressive strength of concretes containing 10% and 15% crumb rubber. It should be noted that in the case of using nanosized materials, particularly silica fume in concrete, water shortage occurs, which reduces the fluidity because of high surface area. Hence, the workability of concrete decreases, and the concrete in the mold does not compact properly. The other mineral additive that is used in rubberized concrete is fly ash. The use of both rubber particles and fly ash increases workability of the rubberized concrete (Bala et al., 2014). Addition of fly ash over to 20% as a replacement of cement decreases the compressive strength. The results using fly ash mixtures show a slight increase in strength.

The addition of metakaolin shows the greatest increase in the water reducer chemical admixture demand when compared with the control mixtures for the same workability. The addition of metakaolin enhances the stability and viscosity of fresh concrete mixtures, especially in rubberized self-consolidating concrete. The metakaolin in rubberized concrete also shows pozzolanic effect with calcium hydroxide. A significant increase is obtained in 28-day compressive strength by 44% on average when compared to control concretes (Ismail and Hassan, 2015).

In some studies, the use of multiple supplementary cementitious materials has also been investigated in order to enhance the mechanical properties of rubberized concrete. For example, Azevedo et al. (2012) used fly ash and metakaolin in rubberized concrete. It was found that the synergetic effect between fly ash and metakaolin minimizes the strength loss associated with the use of rubber waste. The mix with 5% rubber waste and a partial replacement of cement by 15% fly ash and 15% metakaolin has almost the same resistance to sulfuric acid attack of the reference mix. The synergy between limestone powder and silica fume also caused significant improvement in compressive and tensile strengths in mixtures containing crumb rubber and silica fume (Onuaguluchi and Panesar, 2014).

23.8 Utilization of waste rubber in construction applications

The rubber-waste additives in rubberized concrete act as an absorber and balance all internal stresses in concrete. Rubberized concretes may be used in highway construction as a (i) shock absorber, (ii) in sound barriers as a sound absorber, and (iii) also in buildings as an earthquake shock-wave absorber. However, more significant research is needed before strong recommendations can be made (Topçu and Özçelikörs, 1991; Topçu and Avcular, 1997b).

The addition of crumb rubber to modify bitumens is widely used to decrease the environmental pollution of scrap tires and to develop high-quality hot bituminous mixes. There are two methods used to incorporate waste tire rubber into asphalt—wet processes and dry processes.

Wet process: In wet process, the ground tire rubber is first combined with the asphalt binder and digested before the binder is used to create the hot-mix asphalt. There are two types of wet process, terminal blend (or field blend) and wet process high viscosity. Terminal blends are produced by blending crumb rubber and other additives with bitumen at the supply terminal. The rubber has to be finely ground (≤300 μm) so that it can be easily absorbed by the binder and can be kept in suspension by the limited mixing available from tank circulation. Generally, these binders contain less than 10% by weight of rubber, as higher concentrations are more prone to rubber particles separating from the bitumen. Wet process high viscosity binders are produced adjacent to a hot-mix asphalt production facility using specialized equipment. Current best practice has crumb rubber added to the bitumen in a blending unit and then pumped into a reaction tank fitted with paddles to ensure the crumb rubber remains in suspension. During this process, the bitumen is heated to a high temperature (176–226°C) to facilitate reaction with the rubber, and it is held at a temperature of 150–218°C within the reaction tank. After a period of 45–60 min, the resulting binder is moved to a storage vessel, which also contains agitators to keep the rubber suspended.

Dry process: A cheaper alternative is the dry process, where the aggregate and the crumb rubber are hot-mixed prior to the addition of the bitumen, allowing a maceration time for the mixture until the bitumen–rubber interaction is completed (Hernández-Olivares et al., 2009). The ground tire rubber is used to replace a fraction of the aggregate within the hot-mix asphalt. The crumb rubber is added to hot bitumen and the mixture is shaken until the bitumen–rubber interaction occurs. After that, the aggregate is added to the mixture.

In industry, higher amounts of rubber-tire waste can be utilized as fuel, pigment soot, in bitumen pastes, for roof and floor covers, and for paving materials.

Other potential advantages of rubber derive from its sound absorption capacity as well as its thermal properties. However, the value added by the use of rubber particles is usually insufficient to offset the loss in value as a tire. It has also been proposed to exploit the energy absorption potential of rubber with the production of shock-absorbing elements (Meyer, 2009).

23.9 Future trends

Previous researches have shown that the addition of crumb rubber can significantly improve the thermal insulation of concrete panels by different amounts depending on the size and quantity of crumb rubber. Rubberized concrete insulation panels can be developed for energy efficiency in buildings. A series of separate studies also found that rubberized concrete has sound absorption and noise reduction coefficients on overall 36% (Najim and Hall, 2010). Therefore, new research can be carried out on coating of highway and road barriers with crumbed-rubber particles. Also, rubber particles can be used in the interior of tunnels to reduce traffic noise. It can be widely used for development-related projects such as roadways or road intersections, recreational courts and pathways, and skid-resistant ramps. With this new property, these concretes can be used in architectural applications such as nailing concrete, where high strength is not necessary, wall panels that require low unit weight, and construction elements and barriers that are subject to impact.

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Sustainability of sewage sludge in construction

24

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24.1 Introduction

Recent environmental and engineering research focus on the application of sustainability and sustainable environment to improve waste-management practices in place, and to reduce high energy levels that are currently consumed in waste treatment. A number of international studies were undertaken to investigate the possibility of obtaining effective alternatives to the traditional ways of treating and disposing waste. Alternatives comprise utilising waste products in the construction industry for the production of sustainable construction materials (Khatib et al., 2006; Khatib, 2008).

Prior to the implementation of the European Union Urban Wastewater Treatment Directive (91/271/EEC) of 31 Dec. 1998, around a quarter of raw sewage sludge (RSS) produced in the United Kingdom was either discharged to surface waters via pipes or disposed from ships at sea (Defra, 2012). Discontinuing this route together with the higher standards required by the European wastewater directive, generated significant quantities of sewage-sludge. This added greater challenges for both environmental agencies and local authorities. Since then, the traditional methods of agricultural reuse and disposal have had to be replaced by effective alternatives to improve waste-management practices currently in place, and to meet the directive's deadlines. Alternatives include the utilisation of sewage-sludge products in the construction industry for the production of sustainable construction materials.

24.2 Raw sewage sludge

RSS is a residual stream of suspended/dissolved organic and inorganic materials that results from the primary and secondary treatment processes of municipal wastewaters. RSS is usually in the form of liquid or semisolid liquid that typically contains, depending on operation and processes applied, from two to eight present solids by weight (Metcalf & Eddy Inc. et al., 2003). In wastewater treatment plants, RSS is collected mainly from primary settlement tanks, which are large round or rectangular tanks where heavier particles are allowed to settle to the bottom and to be later swept by scrapers to a submerged outlet. The settled stream is pumped, in the form of thick slurry, to the sludge storage and treatment unit for further processing. RSS may also be collected from secondary and tertiary settlement tanks. Fig. 24.1 shows the typical wastewater treatment processes.

Properties of RSS vary depending mainly on collection seasons, as well as on applied treatment processes to source wastewater. Table 24.1 shows the typical chemical composition and properties of RSS.

Sustainability of Construction Materials. http://dx.doi.org/10.1016/B978-0-08-100370-1.00024-X

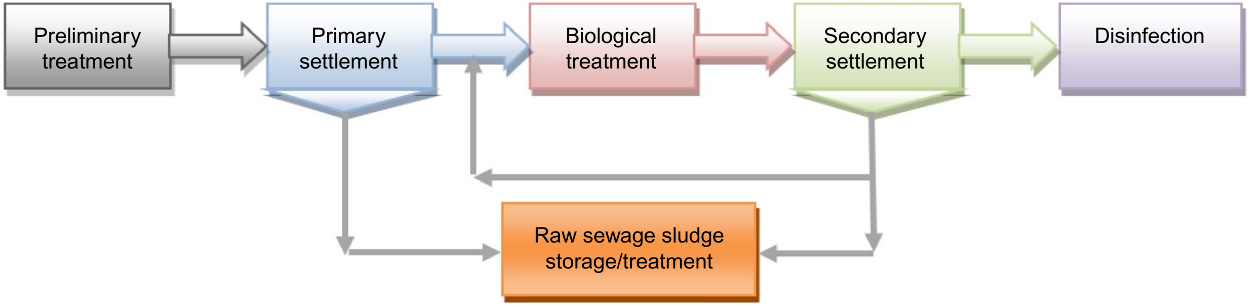

Fig. 24.1 Typical wastewater treatment processes.
Based on Metcalf & Eddy Inc., Tchobanoglous, G., Stensel, H.D., Burton, F.L., 2003. Wastewater Engineering: Treatment, Disposal, Reuse. McGraw-Hill, London.

Table 24.1 **Typical chemical composition and properties of raw sewage sludge**

Item	Range	Typical
Total dry solids (TDS) %	5–9	6
Volatile solids (% of TS)	60–80	65
Grease and fats (% of TS)	6–35	–
Protein (% of TS)	20–30	25
Nitrogen (N, % of TS)	1.5–4	2.5
Phosphorus (P_2O_5, % of TS)	0.8–2.8	1.6
Potash (K_2S, % of TS)	0–1	0.4
Cellulose (% of TS)	8–15	10
Iron (not as sulphide)	2–4	2.5
Silica (SiO_2,% of TS)	15–20	–
PH	5–8	6
Alkalinity (mg/L as $CaCO_3$)	500–1500	600
Organic acids (mg/L as HAc)	200–2000	500

Data from Metcalf & Eddy Inc., Tchobanoglous, G., Stensel, H. D., Burton, F.L., 2003. Wastewater Engineering: Treatment, Disposal, Reuse. McGraw-Hill, London.

24.3 RSS treatment

At sewage-sludge storage and treatment units, further biological, chemical, and physical processes are applied to reduce water content and to eliminate potential associated hazards. Hazards include high, heavy-metal contents, presence of pathogens, and risks associated with the biodegradation of organic matters (production of flammable gases and unpleasant odours). Treatment processes include preliminary operations, thickening, stabilisation, conditioning, dewatering, heat drying, and other processing and thermal reduction. Fig. 24.2 shows the general sewage-sludge treatment procedures.

Following the procedures shown in Fig. 24.2, RSS can be treated partially or fully depending on the quality of the final product. Treatment level also depends on many other factors including on-site treatment facilities, associated cost, energy consumption, and in-place environmental regulations. Different sewage-sludge products have different biological and physical properties, subject to the level and type of applied treatment. Properties include consistency, texture, colour, odour strength, biological activity, and water content (Table 24.2).

24.4 Sewage-sludge production and management

There are ~35 million tonnes of RSS produced in the United Kingdom each year. These quantities are reduced to 25 million tonnes per year by applying further on site physical and chemical processes (Waste on line, 2010). In 2010, 1.41 million tonnes of dry solids were produced from sewage sludge in England and Wales (Defra, 2012).

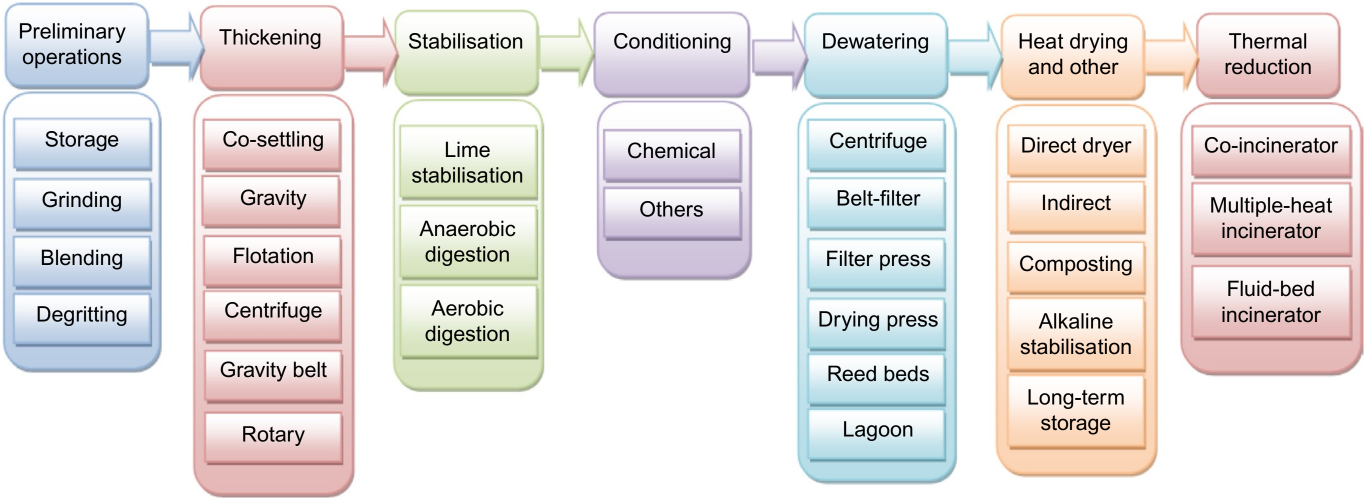

Fig. 24.2 Generalised sludge-processing flow diagram.
Based on Metcalf & Eddy Inc., Tchobanoglous, G., Stensel, H.D., Burton, F.L., 2003. Wastewater Engineering: Treatment, Disposal, Reuse. McGraw-Hill, London.

Table 24.2 Sewage-sludge forms and properties

Terminology and treatment level	Colour/odour	Consistency	Moisture content % of total weight	Reference
Raw sewage sludge, crude sewage sludge, primary sewage sludge, or untreated sewage sludge	Grey with extremely offensive odour	Thick liquid/ slurry	92–98	Metcalf & Eddy Inc. et al. (2003)
Dewatered sewage sludge, sludge cake, or wet sewage sludge	Dark grey with faint odour	Thick paste	50–85	
Dried sewage sludge, thermally dried sewage sludge, dehydrated sewage sludge, or composted sewage sludge	Dark grey inert material	Dried pellets	5–10	Wang et al. (2009)
Incinerated sewage-sludge ash (ISSA)	Dark inert material	Powder	0	Monzó et al. (1999)

Sewage sludge is widely used in agriculture as fertilisers and soil conditioner for being rich in nutrients, trace elements, and organic matter. It improves soil condition, saves the energy required for the production of industrial fertilisers, and recycles phosphorus. Sewage sludge can also be used in land reclamation and can be used as fuel in two ways: as dried sludge pellets for a fossil fuel replacement or as burning biogases, produced from digestion process, in engines to generate electricity. Table 24.3 shows the reuse and recycling routes for sewage sludge in England and Wales in three different years.

24.5 Sewage-sludge utilisation in construction and civil engineering applications

24.5.1 *Ceramic and ceramic-tile manufacturing*

Ceramics are inorganic inert materials made of sintering raw-earth resources (clay, quartz, feldspar, stoneware, and porcelain, which is often made from kaolin) at a temperature of up to 1050°C to produce durable and stiff materials that can be used for engineering and other applications. Engineering applications include the manufacturing of ceramic tiles, clay brick, and lightweight aggregate.

Jordan et al. (2005) prepared ceramic-tile body samples made of standard ceramic clay mixed with different proportions of dry, composted sewage sludge (0–10% of total weight). Samples were prepared using uniaxial pressing. Samples were later

Table 24.3 Sewage-sludge management in England and Wales in tonnes dry solids

Reuse or disposal route	Sludge discharge to surface water			Sludge reuse		Sludge disposed			
	Pipelines	Ships	Others	Soil and agriculture	Others	Landfill	Incineration	Others	Total
1992	8430	273,158	–	440,137	3210	129,748	898,000	24,300	997,673
2008	–	–	–	1,241,639	90,845	10,882	185,890	1523	1,530,779
2010	–	–	–	1,118,159	23,385	8787	259,642	2863	1,412,836

Data from DEFRA, 2012. Waste Water Treatment in the United Kingdom-2012-Implementation of the European Union Urban Waste Water Treatment Directive-91/271/EEC. https://www.gov.uk/government/uploads/system/uploads/attachment_data/file/69592/pb13811-waste-water-2012.pdf.

dried in stove and heated in an electric kiln following a standard heating cycle for high-porosity ceramic. Samples were tested for their bending resistance and water absorption. Results showed that the increase of sludge ratio would increase water absorption and would decrease bending strength.

Favoni et al. (2005) investigated the possibility of fully replacing traditional ceramic clay with powder mixes containing thermally dried sewage sludge and steelworks slag. Cylindrical or rectangular samples were pressed and sintered in electric muffle at 1050–1150°C. Results showed an appropriate level of immobilising hazardous substances contained in the original powders, and a fairly good mechanical strength compared to traditional ceramics. These materials can be used for the production of monolithic ceramic bodies where colours are not important for the final product finish.

Montero et al. (2009) conducted research to study the impact of adding dry, composted sewage sludge and marble residues (0–10% sewage sludge and 0–35% marble residue) to standard clay used in manufacturing ceramic-tile bodies. Results showed that this would provide a better sintering of original powders, and this was due to the improvement of reactivity between the residues used, clay minerals and quartz. However, the technical properties of the final ceramic products, incorporating sewage sludge and marble residues, showed an increase in water absorption and a decrease in bending resistance, and, therefore, the total amount of added waste material must be controlled to produce the ceramic tiles to the required quality.

Qi et al. (2010) studied the possibility of using dehydrated sewage sludge (DSS) pellets, which were produced by thermally treating sewage sludge, as a clay replacement for the production of ultralightweight ceramics (ULWC) products. ULWC were prepared under optimum conditions (addition of DSS was 30%, preheated at 400°C for 20 min, and sintered at 1150°C for 10 min). Results showed that samples incorporated DSS pellets gave low bulk density, good waterproof properties, and safe environmental properties when the leaching test was performed.

Cusidó and Soriano (2011) used sewage-sludge pellets, which are normally produced by drying sewage sludge in low-temperature rotatory kilns, as an alternative material for the natural clay to produce lightweight construction ceramics. Pellets were fired in a furnace at up to 1050°C and then tested for engineering and environmental properties. Results showed that ceramics made of sewage-sludge pellets have a low thermal conductivity, an undetectable amount of hazardous material when leaching test was performed, and no toxic emissions detected during the firing process.

Park et al. (2002) prepared glass and glass-ceramics specimens made of incinerated sewage-sludge fly ash with and without 10% of CaO (by weight). The specimens were heated at 760°C for 1 h, and then fired at a range of 1050–1200°C. The addition of CaO decreases the melting temperature, which provides further economic benefits. Glass ceramics fired at 1050°C for 2 h showed a microhardness of 6230 MPa and a bending strength of 92 MPa. Glass ceramics containing large amounts of diopside (1050°C/2 h) generally showed better physical and chemical properties than their anorthite counterparts due to the interlocking microstructures of diopside crystals.

Merino et al. (2007) tested high probes ceramics samples (46 mm height and 23 mm inner diameter) made of incinerated sewage-sludge ashes (ISSAs), clayey additives (kaolin, montmorillonite, and illitic clay), and powdered flat glass. Samples were subjected to a firing temperature of 1000–1200°C. Specimens were then tested for their engineering properties, including water absorption and compressive strength. Results showed that ceramic construction materials could be obtained by firing sewage-sludge ashes (SSAs) only. However, using the additives stated above improved some engineering properties such as compressive strength.

Mixing ISSA in different proportions with pottery and porcelain clay for the production of tiles was investigated by Chen and Lin (2009b). Nano-SiO_2 was added to the mixes to improve engineering properties by expelling excess air. They were pressed by a machine with a vertical pressure of 35 ± 0.5 MPa, producing floor tile specimens measuring $12 \times 6 \times 1$ cm. Thereafter, specimens were fired at kiln temperature of 1000°C and 1100°C. Results showed that the inclusion of ISSA, to some extent, affected the engineering properties (shrinkage, water absorption, and bending strength), and the addition of nano-SiO_2 showed positive influences on the improvement of engineering properties for both tile specimens. Care should be taken in evaluating the ideal proportions of clay and ISSA and kiln temperature.

24.5.2 Lightweight construction materials

Lightweight cement-based construction materials can be produced by partially or fully replacing the natural aggregates with lightweight materials as well as by using natural or chemical additives that normally form air voids when reacting with cement. Thus, a number of studies were internationally undertaken to investigate the possibility of using sewage-sludge products in cement-based mixes and lightweight aggregate for the production of lightweight construction materials.

The possibility of producing lightweight cement-based materials was investigated by Wang et al. (2005). Cement-based mixes containing various ratios of ISSA were tested for their lightweight and engineering properties. The study concluded that ISSA can be used for the production of lightweight materials, as the hydration process generated pores with diameters $<0.1\,\mu m$.

Chiou et al. (2006) prepared spherical particles made from mixes containing sewage sludge (SS), ISSA, water, and a chemical additive. The mixtures were fired at 1150°C and were tested for their lightweight properties such as unit weight. Results showed that the use of 20–30% of sewage sludge gave the most adequate lightweight properties.

Wang et al. (2009) produced lightweight aggregate from firing mixes containing thermally dried sewage sludge (DSS) and a proportion of coal ash (CA) at 1100°C for 3h. Five different mixing ratios of CA to DSS were evaluated (0%, 10%, 18%, 25%, and 32%). The addition of CA would produce small pores size and consequently increase the bulk density and improve compressive strength. Mixes incorporating 18–25% CA produced the best lightweight aggregate quality and showed an adequate efficiency on immobilising heavy metals.

Lightweight aggregate can also be made from mixes containing natural clay, as a main material, and different proportions of dewatered sewage sludge (80–90% water content). Uniform pellets with similar diameter of 5–10mm were made and then sintered at a temperature of 1050–1150°C for 10–20min. The pellets were then tested for their engineering and environmental properties. Results showed that total water absorption for mixes containing sewage sludge was lower than for those without it; also it was shown that no heavy metals were detected when leaching test was performed (Mun, 2007).

Cheeseman and Virdi (2005) studied properties of lightweight aggregate manufactured from ISSA mixed with clay binder. Spherical pellets were formed and were rapidly sintered in a rotary-tube furnace at temperatures between 1020°C and 1080°C. Results indicated the potential for manufacturing high-quality lightweight aggregate using ISSA with relatively simple processing and low sintering temperature.

24.5.3 Soil stabilisation

The potential of utilising sewage-sludge products in the remediation processes of contaminated soil and improving poor soil quality was widely investigated. Studies showed an evident improvement in certain engineering and environmental properties as described in the studies below.

Theodoratos et al. (2000) investigated the effectiveness of using sewage sludge to immobilise heavy metals in a contaminated soil. Different proportions of biologically treated sewage sludge (70% water content) were mixed with soil samples and were tested for toxicity characteristics leaching procedures (TCLPs). Results showed that adding 15% of total weight sewage sludge reduces Pb, Zn, and Cd solubility by 84%, 64%, and 76%, respectively. The addition of 10% sewage sludge was sufficient to reduce Pb solubility below the US EPA TCLP regulatory limit.

A mixture of ISSA and hydrated lime (L) (4:1, respectively) was used as additive to stabilise a subgrade cohesive soft soil. Five different soil mixes incorporating different proportions of ISSA/L, 0%, 2%, 4%, 8%, and 16% of total soil weight, were prepared, and were tested for their unconfined compressive strength and triaxial compression. Results indicated that samples containing ISSA and hydrated lime showed higher unconfined compressive strength of three to seven times more than that of untreated soil. The swelling behaviours were also reduced. Results of triaxial compression test revealed that shear strength parameter rose with the increase of the additives (Lin et al., 2007).

A mix of ISSA and cement (C) (4:1, respectively) was added to subgrade cohesive soft soil samples in an attempt to improve some engineering properties such as bearing capacity and swelling. Five soil mixes incorporating various ratios of ISSA/C, 0%, 2%, 4%, 8%, and 16% of total soil weight, were prepared, and were tested for their unconfined compressive strength and triaxial compression. Results showed that unconfined compressive strength for samples incorporated ISSA/C was approximately improved by three to seven times more than that of the untreated soil sample. Furthermore, swelling behaviour was also reduced as much as 10–60% for treated samples (Chen and Lin, 2009a).

24.5.4 General civil engineering applications

The use of ISSA as an alternative absorbent to fly ash and blast-furnace slag (used in wastewater treatment) was investigated by Pan et al. (2003b). Results showed that ISSA could be efficiently used as an absorbent for copper removal from wastewater with a removal efficiency of >98%.

Zhao et al. (2009) investigated the potential use of ceramic particles made of firing dried sewage sludge, fly ash, and sand (1:1:1 by weight) in wastewater treatment process. Produced ceramic particles were used, in this study, as an alternative option to the ceramic particles made of traditional clay. Results showed that the removal efficiencies of chemical oxygen demand and ammonium nitrogen, in sewage sludge–fly ash ceramic particles reactor, were all higher than those of traditional clay ceramic particles reactor.

A study was undertaken to examine the possibility of using bottom ashes (resulting from burning dried sewage sludge) as an alternative material to the natural clay used in landfill lining construction. Samples were prepared and tested to determine particle-size distribution, Atterberg limits, compaction characteristics, hydraulic conductivity, and shear-strength parameters. Results of this study showed that properly compacted and stabilised sewage sludge ashes have the required properties to be used in landfill covers or liners (Okol and Balafoutas, 1998).

24.5.5 Cement-based systems

Solidification–stabilisation (SS) technology was applied to sewage-sludge products by mixing them with various binding materials. SS technology refers to a group of cleanup methods that prevent or slow the release of harmful chemicals present in contaminated materials. These techniques usually do not change the chemical composition, but keep pollutants from moving into the surrounding environment. This involves binding the hazardous substances and cement to form a solid block where pollutants are encapsulated and trapped in a hardened mass. This approach has been utilised recently by waste-management professionals, environmentalists, and engineers to both treat hazardous substances and produce useful materials that can be used for other applications. A number of studies were therefore undertaken to examine the effectiveness of applying the solidification–stabilisation technology to sewage-sludge products for the production of sustainable construction materials. Sewage-sludge products included ISSA, wet sludge, dewatered sludge, and dry sludge.

ISSA is an inorganic material resulting from the incineration of sewage sludge (dewatered or thermally dried) that arises from municipal wastewater plants. The total quantity of ISSA produced in the United Kingdom is ~100,000 tonnes per annum (Dunster, 2007). The application of ISSA as cement replacement in cement-based materials was investigated to further understand the impact of partially replacing the main binding materials.

Monzó et al. (1999) used ISSA (15% and 30% of total weight) as a cement replacement in mortars. Prepared samples were tested for their engineering properties, including relative compressive gain and flexural strength gain. The results revealed that the SSA behaves as an active material, resulting in high compressive-strength values in comparison with the control mixes, probably due to the pozzolanic properties of used SSAs (Wild et al., 1995).

Different proportions of ISSA (10–30% of total weight) were used by Garcés et al. (2008) as a cement replacement. Engineering properties were tested including workability, compressive strength, porosity, and length change. Results showed that the best compressive strength was obtained when 10% substitution was used.

Mortars incorporating a higher proportion of SSAs (25% and 50% cement replacement) were investigated by Cyr et al. (2007), and results showed that the addition of SSA induced short delays of cement hydration and lower compressive and flexural strength. It was also shown that presence of SSA has long-term positive impacts, which might be related to its pozzolanic properties (Mangat and Khatib, 1995). The amount of elements leached from samples incorporating SSA was slightly higher than from the reference mixes but with the same order of magnitude.

The workability for mixes containing SSAs (0–30% cement replacement) was investigated by Monzó et al. (2003). Flow-table spread was measured for mortars with various SSA content. Results showed that the inclusion of SSA decreased the workability, and this was due to the irregular morphology (shape nonspherical) of SSA particles and the high water absorption on SSA particle surfaces.

A study was carried out by Pan et al. (2003a) to investigate the influence of SSA fineness on initial and final setting time, workability and compressive strength. Paste samples, incorporating 20% of SSA (fineness of 500–1000 m^2/kg) as a cement replacement, were prepared for this purpose. The results showed that mixes containing finer SSA had longer setting time, better workability due to lubricant effects and morphology improvement, and higher compressive strength.

Liu et al. (2011) tested unfired brick samples made of dewater sewage sludge (50% water content) and four binders (Portland cement, ground silica cement clinker, alumina cement, and slag cement). Samples with different cement/sewage sludge/water ratios were prepared and tested for engineering and environmental properties. Tests included compressive strength, freeze and thaw, and leaching test. Results revealed that the compressive strength of samples containing alumina cement was higher than samples with other types, and compressive strength loss, due to freeze and thaw process, was also less than the other types. Safe levels of heavy metals leaching were observed.

Valls (2000) tested mixes containing wet sewage sludge (68% water of total weight), Portland cement, and coal fly ash for their initial and final setting times. The study mainly concluded that the greater the proportion of sewage sludge, the greater the delay in initial and final setting times, as well as the addition of coal fly ash, increased setting time.

Valls (2002) investigated various mixes containing wet sewage sludge (68% water content), Portland cement, sand, and water with either fly ash or an accelerating agent ($CaCl_2$). Prepared mixes were tested for their engineering and environmental properties. Results generally showed that the greater the amount of sewage sludge, the less the values of compressive strength, with higher compressive strength values for samples without fly ash compared with those containing it. Additionally, results showed a high degree of heavy metal retention in all mortar mixes, which ranged between 84% and 100%.

Malliou et al. (2007) prepared mortar mixes incorporating wet sewage sludge (74% water content), Portland cement, sand, calcium chloride, and calcium hydroxide. Specimens were tested for their engineering and environmental properties. Results showed that as the amount of sewage sludge increases, the compressive strength values decreases. It was also noted that samples containing calcium chloride (this is normally used as an accelerator) had an improved compressive strength, and the best results were observed for samples containing 3% $CaCl_2$ and 2% $Ca(OH)_2$. The inclusion of sewage sludge prolonged setting time, and therefore it was recommended to add acceleration agents to reduce setting time. The results showed high degree of heavy metal retention, which ranged between 0% and 100%, with best results given from mixes containing sewage sludge, 3% $CaCl_2$, and 2% $Ca(OH)_2$.

Jianli et al. (2010) used magnesium oxychloride cement (MOC) as a main binder to stabilise wet sewage sludge (85% total water content). Five mixes incorporating different ratios of the above materials were prepared. Testing included the unconfined compressive strength after 10-day curing time, initial and final setting time, and toxic leachability. The study showed that the compressive strength improved constantly with the increase of MOC, with best result obtained for mix containing 20:100 MOC/sludge, respectively. Also the initial and final setting times were shorter after increasing the amount of MOC. The study also concluded that heavy metal retention capability improved with increasing the proportion of MOC.

A study was undertaken by Cheilas et al. (2007) to investigate the effectiveness of applying solidification/stabilisation technology to wet sewage sludge (78% water content) by mixing it with Portland cement, sand, and jarosite/alunite (JA). Two curing types were applied, traditional and autoclave treatment (16 bar for 3 h at 200°C). Samples were then tested for their engineering and environment properties. Results showed that compressive strength for samples with JA was generally less than for samples without it. The results also showed high degree of heavy metal retention in both samples (with and without JA) when TCLP was performed.

Katsioti et al. (2008) used bentonite/cement mortar to stabilise/solidify wet sewage sludge (66.5% water content) contaminated with heavy metals. Various mixes containing sewage sludge, cement, sand, and bentonite were prepared and were tested for their engineering and environmental properties. Results showed that compressive strength was significantly affected by the addition of bentonite, as samples without bentonite showed higher compressive strength. The results also showed high degree of heavy metal retention when TCLP was performed.

Physical and mechanical properties of concrete mixes with sewage sludge (15% water content) were determined. Four different mixes incorporating Portland cement, sand, coarse aggregate, and water with different ratios of dry sewage sludge (0%, 2.5%, 5%, and 10% of total binder weight) were tested for their engineering properties. Results showed that samples with more sewage sludge proportion gave less density, less compressive strength, and less flexural strength (Valls et al., 2004). Durability properties for concrete samples containing materials stated above were also examined. A number of durability tests were performed including combined wet–dry cycles using fresh water, seawater, and water containing 5% sulphates. Accelerated ageing in an autoclave and accelerated carbonation were also performed. The study concluded that samples with sewage sludge showed acceptable and comparable results to the reference samples (Yagüe et al., 2005).

24.5.6 RSS as a water replacement in concrete and mortar mixes

Due to the fact that it contains about 97% water of total mass, RSS was suggested for use as a water replacement in cement-based materials (Hamood and Khatib, 2012a,b,c). A number of concrete and mortar mixes that contained RSS as a water replacement were investigated for their physical, mechanical, environmental, and durability properties. Cement was partially replaced with 0–20% unprocessed fly ash by total binder mass. Higher proportions of unprocessed fly ash (40–80% of total binder mass) was also investigated (Hamood et al., 2013a,b,c,d).

The outcomes of the investigations were encouraging in that the cement-based materials containing RSS and unprocessed fly ash that were produced demonstrated relatively good engineering, durability, and environmental properties in comparison with the control mixes. The inclusion of unprocessed fly ash significantly reduced flowability/workability. This may be due to the presence of the coarse materials (>45 μm) in the unprocessed fly ash, as workability decreases with increasing the coarse proportion in fly ash (Owens, 1979). An additional factor that influenced the workability is the loss on ignition value, as it is related to the unburned carbon amount in fly ash. The porous carbon particles absorb hydration water resulting in less workability (Minnick et al., 1971; Brink and Halstead, 1956; Welsh and Burton, 1958; Rehsi, 1973). However, the inclusion of unprocessed fly ash improved long-term compressive strength for both mixes with RSS and water. The best compressive strength results were recorded when cement was replaced with 10–20% unprocessed fly ash by weight of total binder. The results also showed that sulphate attack resistance improved when fly ash was included (Mangat and Khatib, 1995). Moreover, safe concentration levels of heavy metals and free ions were detected when leaching test was performed. However, it must be kept in mind that more environmental tests must be performed before any large-scale use is undertaken (Hamood, 2014).

24.6 Future trends

In addition to the production of sustainable construction materials, the outcome of utilising sewage products in the construction industry could see huge financial savings to the current economical constraints by eliminating the costly processes involved in treating these wastes. This would also lead to a huge reduction in energy consumption. Furthermore, there are huge environmental benefits from the prevention of RSS transportation to landfills and incinerators.

With the continuous and unsustainable increase in the world population, there will be a greater demand for freshwater and more materials. RSS may provide an opportunity to save freshwater by using it in applications such as construction. Also, other waste materials can be incorporated in a cement-based system containing RSS. These wastes include various types of coal fly ash (low and high carbon), slag, ground brick, and flue gas desulphurisation residues. There can be adequate uses of these wastes in cement-based materials (Wild et al., 1995; Khatib and Wild, 1998; Khatib et al., 2006). Also, recycled materials such as crushed concrete, brick, and foundry sand can be used as aggregate replacement in cement-based applications with RSS as the water replacement. However, more research is required on the utilisation of RSS in construction materials. More mechanical, durability, and environmental properties of construction materials need to be thoroughly investigated when RSS is present. For example, the effect of bacterial growth with the duration of exposure can be an area for further investigation when RSS is present. However, the overall benefits of using RSS in useful and high-grade applications can outweigh the expensive treatment (ie, wastewater treatment) that would normally be used.

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Sustainability of gypsum products as a construction material

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25.1 Introduction

Gypsum remains one of the most common mineral binders. Gypsum-based products are known as being environmentally friendly materials. The main component of the products is calcium sulphate, which exists in hydrous and non-hydrous compounds: dihydrate ($CaSO_4{\cdot}2H_2O$), hemihydrate ($CaSO_4{\cdot}0.5H_2O$) and anhydrate ($CaSO_4$). Compared to lime and cement-based materials, gypsum has a neutral pH. Gypsum is normally white, which means that gypsum products have desirable decorative properties. Gypsum binders are energy-saving materials because calcination processes occur at significantly lower temperatures than they do with cement and lime, usually within a range of 135–180°C. For this reason fuel consumption will be lower and CO_2 emissions from gypsum manufacturing are minor. Because of rapid setting and hardening, gypsum products do not require accelerated curing. They are lightweight and have excellent mouldability, fire resistance and sound resistance. However, gypsum products have low resistance to water. Application of chemical admixtures and mineral additions, cement or lime addings, reinforcing elements permits to achieve varied properties for gypsum binders.

Gypsum offers many advantages as a building material because of the performance of its properties. Prefabricated products (dry wallboards, ceiling panels, flooring panels) form fire-resistant interiors (Greene, 2009; Sampson, 2011). Gypsum products are nontoxic and are highly soundproof, heat-insulating materials. Gypsum blocks and panels are used in partitions and internal walls instead of ceramic bricks, cellular concrete blocks and other relevant products. Gypsum can be a substitute for lime and cement in interior plasters.

Physically gypsum is infinitely recyclable; however, the recycling process requires additional energy. Gypsum wastes, mostly from pure gypsum, can be used in construction, agriculture and other industrial areas.

Synthetic gypsum, made of chemical by-products, is used widely for the production of gypsum products. Partially or completely, synthetic gypsum can be a substitute for natural gypsum and as cement admixtures, gypsum-based plasters, drywalls and more. Growth of the use of gypsum-containing wastes reduces the load in landfills.

All aspects prove that gypsum products are considered to be sustainable materials.

Sustainability of Construction Materials. http://dx.doi.org/10.1016/B978-0-08-100370-1.00025-1

25.2 Types of gypsum products

25.2.1 Gypsum binders

25.2.1.1 Raw materials

Gypsum binders are inorganic, air-hardening mineral materials. Both natural and synthetic raw materials can be used for manufacturing binders. Their distinctive feature is the presence of hydrous calcium sulphate $CaSO_4 \cdot 2H_2O$ (gypsum) or anhydrous calcium sulphate $CaSO_4$ (anhydrite).

Natural *gypsum*, shown in Fig. 25.1, is the most common raw material for production of binders. It is a soft mineral, attributed to sedimentary rocks, normally grey or with no colour, with a humidity of about 8–10%. Gypseous stone, a denser modification of gypsum that is composed of gypsum and sandy and clayey stone, is normally used to produce gypsum binders. Gypsum deposits are widely distributed, and reserves are considered to be large (Fig. 25.2). According to U.S. Mineral Commodity data (MCS, 2015), 90 countries produced gypsum in 2014 with a total production of 246 million tonnes. China maintains its position as the world's leader in gypsum mining. Thus the rates of depleted reserves of gypsum differ from country to country. The data (MCS, 2015) permit the conclusion that, by maintaining the 2014 level of domestic mine production, which in most of the countries studied changed insignificantly as compared to 2013, found reserves could be sufficient for the next 20 years in India, 40 years in the United States and Poland and 250 years in Canada.

Gypsum is extracted either by open-pit quarrying or by underground mining. Open-pit quarrying is more common and normally less energy- and resource-consuming. For underground gypsum mining, because of horizontal laying of the mineral, a room and pillar mining method is used. Primary mechanical operations include drilling, blasting, excavating, primary crushing and screening (Bushi and Meil, 2011).

The occurrence of *anhydrite* (Fig. 25.3), anhydrous calcium sulphate $CaSO_4$, is rather rare. Anhydrite usually forms underlays in gypsum deposits. Anhydrite has

Fig. 25.1 Natural gypsum from Western Ukraine.

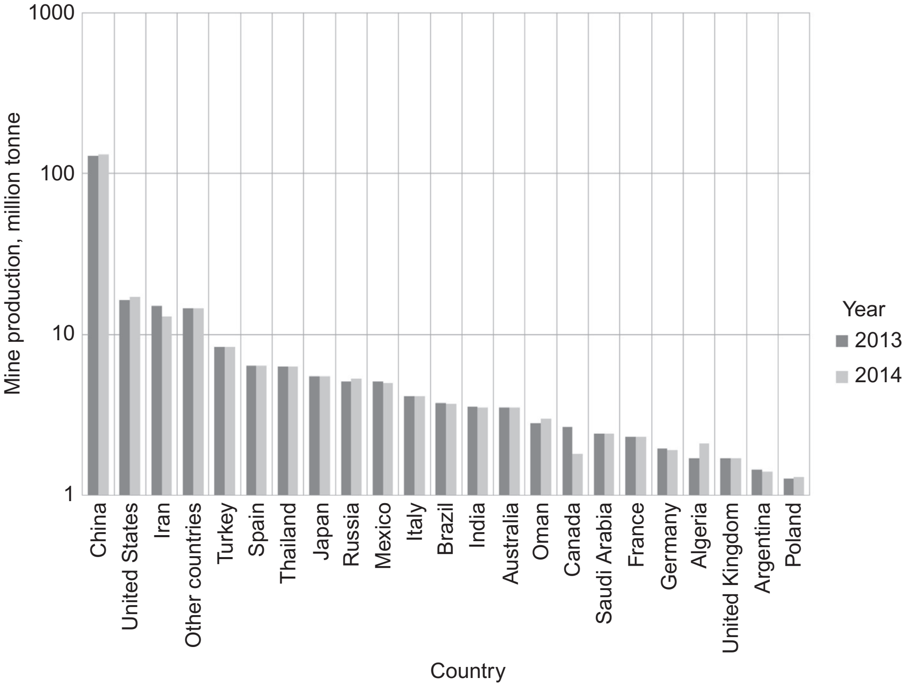

Fig. 25.2 World mine production of gypsum.
From MCS, 2015. Mineral Commodity Summaries: U.S. Geological Survey, 196 pp. Available at: http://minerals.usgs.gov/minerals/pubs/mcs/2015/mcs2015.pdf.

Fig. 25.3 Anhydrite from the Naica Mine, Chihuahua, Mexico. Harvard Museum of Natural History.
Data from http://commons.wikimedia.org/wiki/File:Anhydrite_HMNH1.jpg.

Table 25.1 **Physical properties of gypsum and anhydrite**

Properties	Gypsum	Anhydrite
Specific gravity (g/cm^3)	2.32	2.89
Mohs hardness	1.5–2.0	3.0–3.5
Ultimate strength (MPa)		
Compression	17	80
Tension	2	7
Brittleness index	8.5	11.0
Fusibility	5	2
Magnetic susceptibility	-0.44×10^{-3}	-0.37×10^{-3}
Dielectric conductivity	5.2–6.2	5.7–7.0

a dense crystal structure because of the absence of crystallised water (Table 25.1), which causes a higher value of mechanical properties.

Normally, the gypsum used in the production of binders should have no less than 80% of $CaSO_4{\cdot}2H_2O$ (or $CaSO_4{\cdot}2H_2O + CaSO_4$).

Synthetic gypsum products (industrial wastes containing a high proportion of calcium sulphate) are applied as a total or partial substitution for natural gypsum. There are more than 50 different types of gypsum wastes. Most of them can be considered as substitutes for natural gypsum (Table 25.2). The most common by-products are phosphorgypsum (Fig. 25.4), flue gas desulphurisation (FGD) gypsum and borogypsum (Table 25.2).

Synthetic gypsum normally consists of calcium sulphate dihydrate, calcium sulphate hemihydrate and anhydrite. Basic chemical reactions for obtaining synthetic gypsum are as follows:

$$\text{Phosphorgypsum: } Ca_5F(PO_4)_3 + H_2SO_4 \rightarrow 5CaSO_4 + HF + 3H_3PO_4 \tag{25.1}$$

$$\text{FGD gypsum: } CaCO_3 + SO_2 + 2H_2O \rightarrow CaSO_4 \cdot 2H_2O + CO_2 \tag{25.2}$$

$$\begin{aligned}\text{Borogypsum: } & 2CaO \cdot 3B_2O_3 \cdot 5H_2O + 2H_2SO_4 + 6H_2O \\ & \rightarrow 2CaSO_4 \cdot 2H_2O + 6H_3BO_3\end{aligned} \tag{25.3}$$

The total amount of the calcium sulphate components vary within a range of 80–98% by weight. Synthetic gypsum may also contain substances from initial raw (ores, acids), silica and other components. Synthetic gypsum requires removing or neutralisation of the impurities. Sometimes it is impossible to remove all the harmful impurities, as they can be inside the calcium sulphate crystals. Some of gypsum containing industrial wastes, like phosphorgypsum, have a rather high level of radioactivity. In many manufacturing processes, synthetic gypsum is segregated in the form of water sludges of fine-grained particles. They have to be dried out and briquetted for transportation, which requires additional energy.

25.2.1.2 Dehydration of gypsum

Gypsum binders are obtained by thermal treatment of gypsum and subsequent grinding. For anhydrite-based binders, only grinding with activating admixtures is required.

Table 25.2 Types of synthetic gypsum

Name of product	Technological process resulting in the formation of synthetic gypsum	References for further reading on properties and application
Phosphorgypsum	Processing phosphate (apatite) ore into fertiliser with sulphuric acid	Dvorkin and Dvorkin (2011)
Flue gas desulphurisation (FGD) gypsum	Flue gas desulphurisation by alkaline sorbent (limestone) at fossil fuel power plants	Tzouvalas et al. (2004), Mangat et al. (2006), Lund-Nielsen (2010), Guan et al. (2011), and Khatib et al. (2013, in press-a,b)
DeSulphoGypsum (DSG)[a]	Liquid gas desulphurisation by alkaline sorbent (limestone) at liquid and gaseous fuels power plants	
Borogypsum	Boric acid production	Boncukcuoğlu et al. (2002) and Sevim and Tümen (2013)
Fluorgypsum (fluoroanhydrite)	Sulphuric treatment of hydrofluoric acid	Magallanes-Rivera and Escalante-García (2014)
Ferrogypsum	Lime treatment of iron vitriol solutions	Norelius (1984)
Titanogypsum	Titanium dioxide pigment production	Bensted (1981) and Claisse et al. (2008)
Red gypsum	Waste product from the titanium dioxide industry with considerable quantity of ferric flocs	Gázquez et al. (2011) and Hughes et al. (2010)
H-acid gypsum	Neutralisation of free sulphuric acid collected during the formation of intermediate dyes	Garg and Jain (2010)
Hydrolysis gypsum	Hydrolysis of lignocelluloses by concentrated sulphuric acid	Bon and Ferrara (2007)
Citrogypsum	Production of citric acid	Kostić-Pulek et al. (2000)
Tartratogypsum	Production of tartaric acid	Bensted (1981)

[a] In many references, FGD and DSG are considered to be synonyms.

Therefore, embodied energy for their production is significantly lower compared to binders obtained by thermal treatment of gypsum. The basic technological operation for obtaining gypsum binders from calcium dihydrate sulphate is a thermal treatment for partial or complete dehydration.

Basic products made of natural and synthetic gypsum are building gypsum, mostly made of β-hemihydrate, high-strength gypsum consisting of α-hemihydrate, Keene's plaster (anhydrite cement), Estrich gypsum (high-temperature calcined gypsum) (Zhang, 2011).

Depending on the type of thermal equipment and parameters of dehydration, different calcium sulphate-based products form. They vary in composition, structure and properties (Fig. 25.5).

Fig. 25.4 Phosphorgypsum, PJSC ‘Rivneazot’ (largest nitrogen fertiliser producer in Western Ukraine).

The dehydration process consists of two stages: first, one with a hemihydrate formation

$$CaSO_4{\cdot}2H_2O \rightarrow CaSO_4{\cdot}0.5H_2O + 1.5H_2O \tag{25.4}$$

and, second, one with an anhydrite formation

$$CaSO_4{\cdot}0.5H_2O \rightarrow CaSO_4 + 0.5H_2O \tag{25.5}$$

All the reactions are endothermic (energy-absorbing). Depending on the equipment applied and technological parameters, α- or β-hemihydrate (α- or β-anhydrite) form. For α-hemihydrate formation, 83.2 kJ/mol is required, for β-hemihydrate, 85.3 kJ/mol is required, when water is evaporated and 17.2 and 19.3 kJ/mol respectively if water condensates in a liquid state. For insoluble anhydrite formation, the dehydration heat is 105 kJ/mol.

The differences between the conditions of manufacturing for α- and β-calcium sulphate hemihydrates cause the differences in their structure and properties. Most researchers think that unequal dispersion of crystals is the main reason for differences in hemihydrates properties (Singh and Middendorf, 2007). The higher true surface area of β-hemihydrate particles and their porous structure cause a greater demand for water and ultimately a lower strength of gypsum. Although particles of α-hemihydrate have fewer defects in crystalline lattice than β-hemihydrate, they are less porous and the size of the particles is about five times smaller. For these reasons, the water demand is lower and the final strength is significantly higher. Properties of α- and β-anhydrites are similar to relevant hemihydrates (Odler, 2000).

Intermediates form in the process of anhydrite production. Dehydrated (dry) hemihydrates form as the result of remaining crystalline water; they have higher water demand (about 5–6%) than hemihydrates; however, their hardening rate and strength are similar to hemihydrates. Some researchers dispute the existence of these modifications.

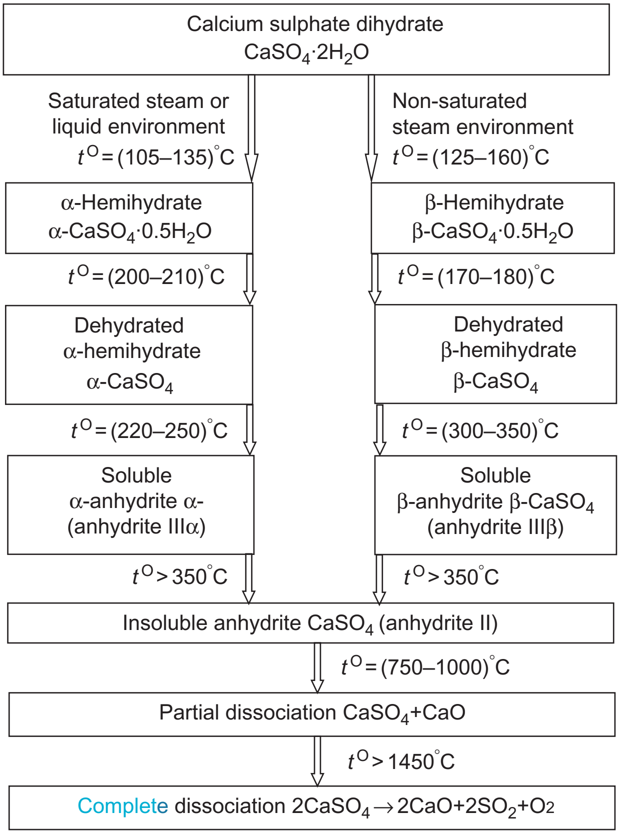

Fig. 25.5 Transformation process of $CaSO_4 \cdot 2H_2O$ at heating according to Belyankin and Berg. Modified from Dvorkin, L., Dvorkin, O., 2011. Building Mineral Binding Materials. Infra-Ingeneria, Moscow, pp. 27–90 (in Russian).

Soluble anhydrite has a higher water requirement than hemihydrate (25–30%) and subsequently lower strength. At temperatures above 350°C, insoluble anhydrite forms; it does not react with water and does not harden. Differences between α- and β-calcium sulphate hemihydrates structure and properties are shown in Table 25.3.

25.2.1.3 Manufacturing of gypsum binders

The basic manufacturing operations of gypsum binders are preliminary preparation of the raw gypsum (crashing, drying and fine grinding), thermal treatment and additional grinding when required. At each stage of the technological procedures, automatic control should be provided.

β-Hemihydrate binder

β-Hemihydrate (also known as plaster of Paris) is the most commonly used gypsum binder in construction. It is produced in an apparatus connected to air. Different equipment is used, depending on the required quality of finished product and its subsequent

Table 25.3 Differences between α- and β-calcium sulphate hemihydrates structure and properties

Parameter	α-Hemihydrate	β-Hemihydrate
Method of preparing	Wet (autoclaving)	Dry (calcining)
Structure (X-ray analysis)	No significant differences	
Structure (scanning microscope)	Well-formed transparent idiomorphic crystals with sharp crystal edges	Flaky particles made up of small crystals
True surface area	Lower	Higher
Defects of crystals	Fewer	More
Water demand	Lower (40–50)%	Higher (60–70)%
Strength	Higher	Lower

Based on Odler, I., 2000. Special Inorganic Cements. E&FE Spon, Modern Concrete Technologies Series, 11. Calcium sulfate based binders, pp. 191–201; Singh, N.B., Middendorf, B., 2007. Calcium sulphate hemihydrate hydration leading to gypsum crystallization. Prog. Cryst. Growth Charact. Mater. 53, 57–77; Dvorkin, L., Dvorkin, O., 2011. Building Mineral Binding Materials. Infra-Ingeneria, Moscow, pp. 27–90 (in Russian).

application. Dehydration processes can be conducted in kettle calciners, rotary dryers or roller mills, when grinding and calcining are combined.

Kettle calciners are applied extensively in typical gypsum manufacturing processes (Reichardt and Sander, 2006; Dvorkin and Dvorkin, 2011). Before calcination in kettle calciners, gypsum is subjected to crushing, drying and grinding to obtain particles less than 2 mm in size. In kettle calciners, gypsum is heated outwardly by hot gases passing through flues. Such indirect heating ensures a constant temperature; overburning and contamination of gypsum with combustion products are avoided. The duration of calcination depends on preliminary drying and grinding rates and varies within 1–3 h, and the temperature of dehydration is 150–180°C. Equivalent fuel consumption is 30–40 kg per 1 tonne of hemihydrate. To reduce the temperature and water demands of gypsum binding, a saturated solution of NaCl can be added (0.1% by gypsum weight).

Kettle calciners can operate in either batch or continuous mode. Batch mode calciners are easy to maintain and also in terms of controlling the calcination regime; thus they have more complicated systems of dust collecting and low productivity because of the pauses.

A continuous mode increases productivity. Because of the air's constant, high saturation of water vapour, the structure and properties of gypsum binders are uniform. Constant mixing ensures a homogeneous product. To minimise energy consumption, exhaust gases are used in the gypsum pre-drying process.

Rotary dryers are applied at highly productive technological processes. These dryers work in continuous mode. The contact with gases can be direct or indirect (Fig. 25.6). Pieces up to 35 mm in size burn. Equivalent fuel consumption for a rotary dryer is 40–50 kg per 1 tonne of hemihydrate. When calcination is finished, gypsum is ground in ball mills. The ground gypsum is collected in a cyclone.

Energy-efficient manufacturing technologies have been developed which permit combining drying and grinding processes in roller mills or hummer mills, where heating with gas stream and grinding are accomplished simultaneously (Fig. 25.7).

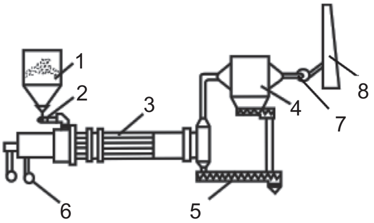

Fig. 25.6 Gypsum production in rotary dryers: Hopper (1); Feeder (2); Rotary dryer (3); Dust collector (4); Conveyer for finished product (5); Ventilator (6); Smoke exhauster (7); Tube for blowing up exhaust gases (8).

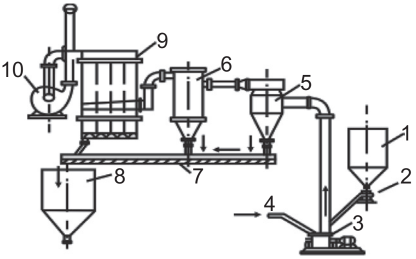

Fig. 25.7 Gypsum manufacturing process at combined calcination and grinding: Hopper (1); Feeder (2); Hummer mill (3); Gas pipeline (4); Cyclone (5); Filter (6); Screw (7); Hopper for finished product (8); Filter (9); Ventilator (10).

When a portion of combustion gas returns to the mill after dust-catch chambers, manufacturing processes may provide recirculation of a heat-transfer agent. Applying gas recirculation leads to an increased consumption of electricity, but it reduces the consumption of fuel.

Crushed gypsum with a particle size up to 15 mm is transported to hopper 1 and then through feed 2 to hummer mill 3, where combustion gases from the fire chamber move through pipe 4. Grounded dehydrated material is transported with gas steam to the cyclone 5.

Drying, grinding and calcining can be performed in heated impact mills. In these mills, hot gas contacts gypsum. It dries and calcines the gypsum and then conveys hemihydrate to a product cyclone for accumulation (CAPEF, 1995).

EM mill, or effective innovative equipment, provides drying, grinding and classifying processes of hemihydrate for further production of gypsum boards. Comparing to traditional roller mills, it contains grinding balls, which move freely between the grinding rings. A gypsum bed forms between these balls and the grinding ring. When the mill switches off, the balls sweep the material away from the mill (Gypsum Technik, 2014).

α-Hemihypdrate binder

α-Hemihypdrate (high-strength gypsum) is obtained by removing crystalline water in liquid state. The most common method is autoclaving with subsequent drying and grinding. The last two processes can be conducted with the same equipment (Fig. 25.8). Hemihydrate drying at atmospheric pressure is substituted for evaporation at a pressure 0.1 MPa.

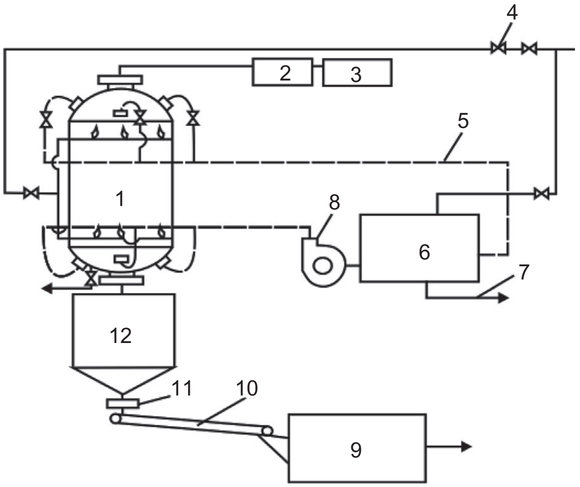

Fig. 25.8 Technological circuit of high-strength gypsum manufacturing with mutual drying and dehydration in one apparatus: Autoclave (1); Screen (2); Crusher (3); Steam pipeline (4); Heat carrier line (5); Heater (6); Pipe for condensation drainage (7); Ventilator (8); Mill (9); Conveyor (10); Disk feeder (11); Hopper (12).

Taking into consideration the significant amount of extracted water, the method of self-steaming can be energy-effective. With that method, saturated vapour does not transfer from the outside; vapour forms because of the release of hydrated water. This equipment is sealed to avoid contact with the environment. Another method is thermal treatment in a liquid environment of water solutions of salts and surface-active substances. In such environments, the transfer of heat is more active, and chemical reactions progress faster. In this case, the technology includes fine grinding of gypsum, thermal treatment in dehydrators, filtration or centrifugation of hemihydrate, washing and drying (Ferronskaya, 2004).

Anhydrite binders

Anhydrite binders are produced by thermal treatment at 600–700°C and the subsequent addition of activators for setting time regulation (lime, dolomite, blast furnace slag). When natural anhydrite is used, the raw materials are subjected to drying and grinding. For high-temperature treatment of fine-grained gypsum dihydrate, flash calciners are used in order to obtain anhydrite (Calix Technology, 2011; CFC, n/a). In a flash calciner, gypsum is in direct contact with hot gases. At the bottom of the calciner, gypsum is accumulated. Heat treatment is conducted at temperatures of over 500°C. Calciners provide complete recirculation of exhaust gases, which enables efficient fuel performance (Fig. 25.9).

Estrich gypsum is high-temperature calcined gypsum obtained at a thermal treatment of dihydrate at 900–1000°C and with subsequent grinding. It consists of calcium sulphate (75–85%) and small amounts of calcium oxide (2–4%) that activate hardening of Estrich gypsum. Its water demand is 30–35%; it sets and hardens slowly (initial setting time is no less than 2 h after mixing).

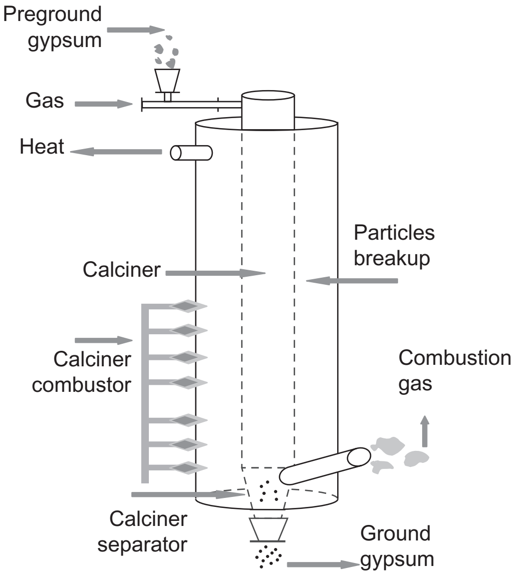

Fig. 25.9 Flash calciner by Calix. Schematic view.
Modified from Catalytic Flash Calcination (CFC) Technology (n/a). http://www.calix.com.au/catalytic_flash_calcination__cfc__technology.html.

Binders made of industrial wastes

At present, the most important and widely used synthetic gypsum products are phosphorgypsum, FGD gypsum and fluorgypsum. Peculiarities of synthetic gypsum processing depend on the initial moisture content, its dispersion and the content of impurities.

Later in this section, we consider the peculiarities of phosphorgypsum processing. Its high specific surface area (3500–3800 cm^2/g) permits the exclusion of crushing and coarse grinding. The phosphorous and fluorous impurities need to be washed and neutralised, which leads to increased fuel consumption. The high content of insoluble anhydrite significantly reduces the binding ability of phosphorgypsum.

Washing out the impurities alone or in combination with neutralisation or sedimentation of the admixtures causes a large volume of polluted water to accumulate (2–5 m^3 per 1 tonne of phosphorgypsum). Additional energy consumption is required for purification. Thermal decomposition of impurities and the introduction of neutralising admixtures before or after calcination are not common because the quality of the binder is unstable.

Different ways to obtain gypsum binders from phosphorgypsum that is intended for the production of gypsum wallboards are shown in Fig. 25.10. According to the first method, phosphorgypsum is washed out and floated to remove water soluble and solid impurities, and then it is subjected to dehydration in the calciner. The second method is applicable for mixtures with a anhydrite to hemihydrate ratio of 1/3 to 2/3. Purified dehydrated phosphorgypsum is subjected to granulation when water and precipitating agents are added and then is cured in the reactor (Fig. 25.10).

The third method obtains phosphorgypsum hemihydrate directly in the manufacturing of phosphoric acid by extraction. As phosphorgypsum contains significantly fewer

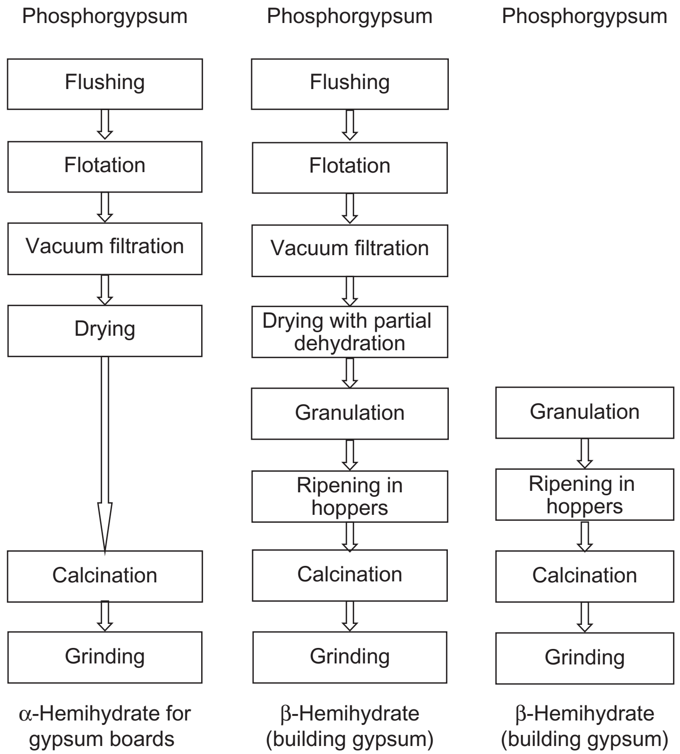

Fig. 25.10 Basic schemes for gypsum binders manufacturing (developed by Knauf).

impurities, it does not require preliminary purification. The technologies of neutralisation of phosphorgypsum by lime is reported elsewhere (Dvorkin and Dvorkin, 2011).

Because of the high-moisture content of synthetic gypsum, autoclaving technologies are most common for obtaining α-hemihydrate (high-strength gypsum). The water remains after filtration evaporates. Heating at 115–125°C, causes crystallisation of α-hemihydrate to occur.

Results of the testing of α-hemihydrate manufactured of FGD gypsum, obtained in hydrothermal conditions at 130°C during 3 h of isothermal curing in rotary autoclave demonstrante that compressive strength of gypsum dry samples reached 67 MPa (Leškevičiene, 2013).

Another effective method for synthetic gypsum calcining is the use of a horizontal impact calciner. It provides calcination of fine-grained material with high-moisture content. Production is up to 100 tonnes/h. Calcining is conducted in one stage (Gypsum Technik, 2014).

25.2.1.4 Energy consumption and emissions of binders manufacturing

The fuel consumption in the production of gypsum binders is compared to the production of lime and Portland cement in Fig. 25.11. It was supposed that 1 tonne of Portland cement's equivalent fuel consumption is 176 kg, its electric power is 108 kWh

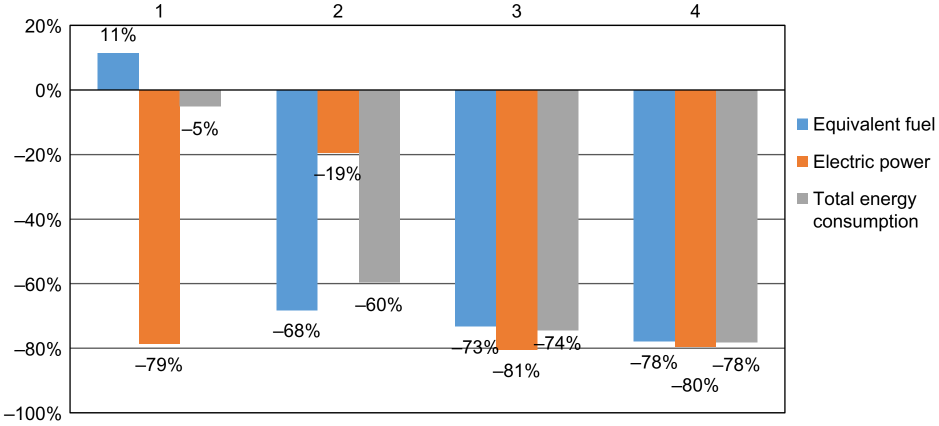

Fig. 25.11 Energy consumption changes per 1 tonne of lime (1); High-strength gypsum made of phosphorgypsum (2); High-strength gypsum made of natural gypsum (3); Building gypsum made of natural gypsum (4).

and its total energy consumption (total equivalent fuel) is 215 kg. All the parameters were considered as 100%, and relative changes for different binders were calculated based on known data (Dvorkin and Dvorkin, 2011). As shown, for production, the unit fuel consumption of gypsum is significantly lower than for Portland cement.

Manufacturing of gypsum binders causes equipment to produce certain emissions. Solid particulate matters (PM) are dominant pollutants in the gypsum manufacturing process. Primary sources of these pollutants are rotary ore dryers, calciners and grinding mills; in the case of finished products, sawing operations create the dominant pollutants. Secondary sources are crushers, screens and stockpiles.

Gaseous emissions caused by fuel combustion contain oxides of carbon (monoxide and dioxide), sulphur and nitrogen. Almost all of the equipment require fuel either for heat treatment (dryers, kilns) or for other processes (crushing, grinding, screening, transporting) (CAPEF, 1995).

25.2.1.5 Properties of gypsum binders

Hardening of gypsum binders occurs according to the exothermic reaction of hydration.

Fundamental reactions of hydration are as follows:

$$CaSO_4{\cdot}0.5H_2O + 1.5H_2O \rightarrow CaSO_4{\cdot}2H_2O \tag{25.6}$$

$$CaSO_4 + 2H_2O \rightarrow CaSO_4{\cdot}2H_2O \tag{25.7}$$

Hydration heat is much lower than that of an endothermic reaction of dehydration (Ferronskaya, 2004):

for α-hemihydrate hydration 17,187 kJ/kg
β-hemihydrate hydration 19,259 kJ/kg
α-anhydrite hydration 25,749 kJ/kg
β-anhydrite hydration 30,187 kJ/kg

Water consumption for building gypsum is 50–70%; for high-strength gypsum, it's 30–40%; for anhydrites, it's 30–35%.

Depending on the *time of setting*, gypsum binders can be divided into groups such as rapid, normal and slow setting. The initial setting time does not occur before 2–6 min, and the final setting no later than 15 and 30 min for the rapid and normal-setting binders, respectively. An initial setting time of 20 min or more is typical for gypsum binders of the third group (Nwaubani et al., 2011a). For building gypsum, the hardening process completes at nearly the 2 h point. The compressive strength for building gypsum, measured in that time, varies from 2 to 7 MPa. For high-strength gypsum, the compressive strength normally varies from 7 to 25 MPa.

Because of rapid setting and hardening, a minimum amount of moulds can be used at conveyor lines for manufacturing gypsum products. For regulating setting and hardening times, retarding and accelerating admixtures are used. Accelerating admixtures are common when recycled gypsum is used; that is, when gypsum that is derived from the waste of gypsum products is used.

The water–gypsum ratio is considered to be the most significant factor affecting gypsum properties (Fig. 25.12). Different numerical dependencies develop the relationship between water content and gypsum strength (Dvorkin and Dvorkin, 2011; Yu and Brouwers, 2011).

Pores, formed because of water evaporation, are mostly capillary. Therefore, casting gypsum has a comparatively high water absorption rate.

To reduce the water–gypsum ratio, high-range water reducers are used. Polycarboxylate ethers with long side molecular chains are considered to be the most effective ones because they provide a significant steric effect on gypsum particles (Peng et al., 2005; Dvorkin and Lushnikova, 2009). The low water resistance of gypsum products is caused by the high solubility of gypsum dehydrate. To improve water resistance, lime is added. Higher water resistance can be achieved by mixing cement

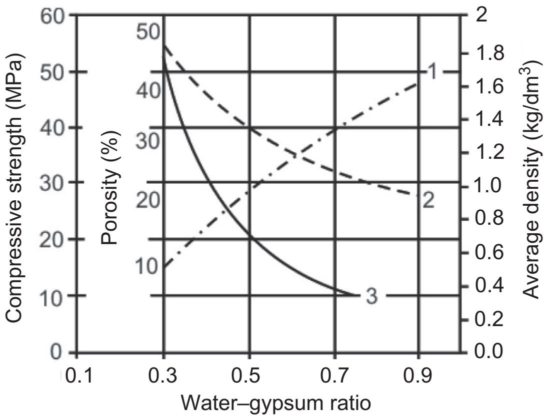

Fig. 25.12 Dependences of basic gypsum properties on the water–gypsum ratio: Porosity (1); Average density (2); Compressive strength (3).
Data from Dvorkin, L., Dvorkin, O., 2011. Building Mineral Binding Materials. Infra-Ingeneria, Moscow, pp. 27–90 (in Russian).

and pozzolanic admixtures (fly ash, blast furnace slag, etc.) (Ferronskaya, 2004; Dvorkin and Dvorkin, 2011; Khatib et al., in press-a,b). Properties vary significantly depending on the humidity of gypsum products. The increase in moisture content leads to irreversible plastic deformations (creep) when load is applied. At the initial hardening period, gypsum expands up to 1% (high-strength gypsum expands up to 0.2%) with subsequent shrinkage within the range of 0.05–0.1%.

Gypsum products have high *durability*. In the case of alternate wetting and drying, the strength of gypsum-based products gradually reduces. Gypsum concrete withstands 30–40 cycles of wetting and drying, and the frost resistance of gypsum plasters and concrete is normally less than 10 cycles. Composite binders, containing cement and pozzolanic admixtures, have high sulphate resistance, because of the low alkalinity of new formations. Gypsum products, when wetting–drying or freezing–thawing is assumed, should have a total porosity of less than 20%. Gypsum-based concrete, when compared to cement-based concrete, is more resistant to carbon dioxide and sulphuric anhydride. Gypsum products have high fire-proofing abilities, which are caused by the evaporation of chemically bound water when exposed to fire. The temperature of materials normally does not exceed 100°C. When porosity increases because of dehydration, heat conductance reduces, which causes a slowing down of the transfer of heat. The fire-endurance period depends on the thickness of the gypsum concrete. For unprotected steel structures, the period is equal to 0.5 h; for reinforced concrete structures, it is 1–2 h; for gypsum concrete, it reaches 3 h and more (Fig. 25.13). Reinforcing with glass fibre increases fire resistance.

Intensive loss of strength starts at temperatures of 400°C and higher because of the amount of dehydration that occurs. Gypsum products do not release toxic substances at fire.

Steel reinforcement in gypsum concrete can be subjected to corrosion because of the neutral pH of gypsum (pH 6–8). Composite binders containing cement and pozzolanic admixtures retard the rate of corrosion in several times; however, steel requires additional protection because reliable passivation is not achieved even in that case.

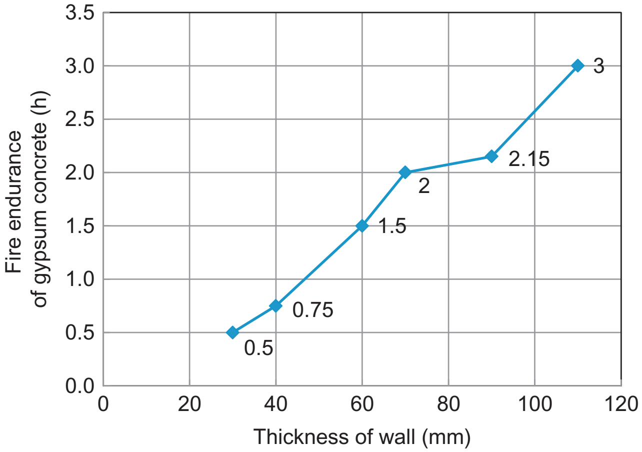

Fig. 25.13 Fire endurance of gypsum concrete, depending on thickness of the wall.

Gypsum expansion at the initial hardening stage also reduces adhesiveness to reinforcements. Significant difference in coefficients of linear thermal extension is an additional reason for poor compatibility of gypsum concrete (20×10^{-6} m/m K) and steel (12×10^{-6} m/m K). Gypsum concrete is normally reinforced with glass, polymer and plant fibre. Aluminium reinforcement is also applicable.

25.2.1.6 Gypsum-based composite binders

To increase the water resistance of gypsum binders, regulate setting time and strengthen parameters and performance, composite binders containing gypsum, lime, cement and pozzolanic admixtures are used. Normally, they are obtained by uniform grinding of all the components in ball mills. Their applications vary depending on composition and properties.

For optimisation of gypsum-based composites, chemical admixtures are applied. They may contain a retarding admixture, a water-soluble polymer, an air-entraining admixture and a superplasticiser (Arikan and Sobolev, 2002). This way low water consumption binders contain chemical admixtures, water reducers and setting retarders. Gypsum composites reinforced with organic and synthetic fibre (wood, glass, polymer) and coarse aggregates (sand, slags, vermiculite, etc.) have advanced mechanical properties. Polymer-modified composite binders have been developed. Adding polymer and the partial replacement of gypsum with Portland cement, blast furnace slag, silica fume and fly ash increases the water resistance of the gypsum composite, improves its strength and reduces its resistance to cracking (Ohama et al., 2000; Magallanes-Rivera et al., 2012). Synthetic gypsum can serve as a component for supersulphated cements as well as for other composite binders (Nwaubani et al., 2011b).

According to its composition and application, gypsum binders and plasters are divided into several categories, as described in Table 25.4.

25.2.2 Gypsum concrete

Gypsum concrete can be normal weight (with an average density 1800–2500 kg/m^3), lightweight (500–1800 kg/m^3) and extra-lightweight (less than 500 kg/m^3). For normal-weight, concrete coarse aggregates (crushed stone and gravel) and fine aggregates (natural sand, screenings of crushed rocks and industrial wastes—slag, fly ash) are used.

For lightweight concrete, porous natural (pumice stone, tuff, shelly limestone) and synthetic materials (vermiculite, clayite, agloporite) are applied as aggregates. There are also common organic aggregates: plant wastes, including wastes from manufacturing timber products and other vegetable raw treatments such as sawdust, cheaps, straw, rush fibre, etc. (Fig. 25.14).

A comparison of the compressive strength of different kinds of gypsum-based concrete is shown in Table 25.5.

The ratio between compressive and flexural strength for normal-weight concrete is 6:8, whereas for lightweight concrete with porous aggregates, it is 9:12. Fibre-reinforcing elements normally have significantly higher flexural strength.

Table 25.4 Types of gypsum binders and plasters

Category	Designation	Type	Designation
A	Gypsum binder	A1	Gypsum binder for direct use
		A2	Gypsum binder for direct use on site
		A3	Gypsum binder for further processing
B	Gypsum plaster	B1	Gypsum building plaster
		B2	Gypsum-based building plaster (≥50% gypsum)
		B3	Gypsum-lime building plaster (>5% lime)
		B4	Lightweight gypsum building plaster (inorganic or organic aggregates)
		B5	Lightweight gypsum-based building plaster (inorganic or organic aggregates)
		B6	Lightweight gypsum-lime building plaster (inorganic or organic aggregates)
		B7	Enhances surface hardness gypsum plaster
C	Gypsum plaster for special purposes	C1	Gypsum plaster for fibrous reinforcement
		C2	Gypsum mortar
		C3	Acoustic plaster
		C4	Thermal insulation plaster
		C5	Fire protection plaster
		C6	Thin-coat plaster, finishing product
		C7	Finishing product

Data from BS EN 13279-1:2008.

Fig. 25.14 Specimen of phosphorgypsum-based concrete with sawdust aggregate.

Table 25.5 Compressive strength of gypsum-based concrete

Binder	Compressive strength (MPa)			
	Normal-weight concrete	Fine-grained concrete	Lightweight concrete	
			Porous mineral	Organic
Building gypsum	4.5–8	2…8	3…9	2.5…4
High-strength gypsum	15…30	3…15	3.5…20	3.5…5
Anhydrite	7…15	2…12.5	3.5…15	3…7.5
Estrich plaster	4.5…8	2…13.5	–	–
Composite binder (water-resistant)	9…35	5…25	5…22	3.5…9.5

Data from Dvorkin, L., Dvorkin, O., 2011. Building Mineral Binding Materials. Infra-Ingeneria, Moscow, pp. 27–90 (in Russian).

Gypsum concrete (mostly based on water resistance of gypsum-based binders) is used for the production of walling blocks and panels in non-load-bearing structures and envelopes. Construction units should be protected from moisture.

Lightweight energy-saving gypsum-based composites with high-performance properties are of primary interest for construction practices. Different existing lightweight gypsum-based composites permit a savings on energy in the production of modules and in the performance of the products.

Research has been done in which gypsum was used to produce self-compacting, lightweight composites with improved thermal and mechanical properties (Yu and Brouwers, 2012). Expanded vermiculite and polypropylene fibres with low thermal conductivity improve the performance of lightweight gypsum composites (Gencel et al., 2014). There are also applied mathematical and computer methods for modelling the gypsum's internal structure formation and for subsequent regulation of the properties to produce high-performance gypsum and gypsum-based materials (Petropavlovskaya et al., 2013). It was determined that granularity regulation and an increase in gypsum fineness are the main factors for manufacturing high-performance gypsum and high-quality gypsum products.

Glass fibre-reinforced gypsum (GFRG) is applied in the production of load-bearing and non-load-bearing wall panels because of an increase in strength and ductility. GFRG has compressive strength up to 70 MPa and higher, flexural strength up to 50 MPa, thermal conductance of about 0.62 W/m K, sound transmission of 40 and a 4 h fire endurance rating that withstood at 700–1000°C (GFRG[1], 2012).

Foam gypsum (Fig. 25.15), consisting of building or high-strength gypsum and water and foaming agents, is used for manufacturing blocks and panels for non-load-bearing walls in framework buildings or low-rise buildings, for insulation of internal walls, for partitions and for acoustic ceilings and heat insulation of camp ceilings.

The average density of foam gypsum varies from 300 to 900 kg/m^3; the strength of foam gypsum varies significantly depending on the strength of the gypsum. The

Fig. 25.15 Laboratory specimen of foam gypsum with an average density of 600 kg/m^3.

compressive strength of foam gypsum specimens with an average density of 600 kg/m^3 in dry state is between 1.5 and 2.5 MPa, when building gypsum strength is 5 MPa, when initial building gypsum strength is 10 MPa, foam gypsum specimens strength increases up to 3.0–4.0 MPa (Ferronskaya, 2004). Depending on the density, thermal conductance of foam gypsum varies from 0.1 to 0.26 W/m K. Keeping the correlation between density and strength requires thorough calculations and non-trivial methods for proportioning, based on physical dependencies which determine the foaming process (Dvorkin et al., 2012).

Recent research has focused on the development of lightweight products containing industrial and urban wastes such as rubber (Rivero et al., 2014), unburnt rice husk, blast furnace slag, modified with polyvinyl alcohol polymer (Khalil et al., 2014), polyamide powder wastes (Gutiérrez-González et al., 2013), insulating composites like cork–gypsum composite for walls or false ceilings (Cherki et al., 2014). There are other advancements in gypsum products, referred to as ‘smart gypsum composites’. They are manufactured with the addition of microcapsules containing phase change materials (PCMs) in order to develop materials with a high-temperature thermal energy storage to be used, for example, in high-comfort-level constructive systems (Borreguero et al., 2014).

25.2.3 Gypsum products

In terms of application, gypsum products can be divided into categories, including walling, heat insulating, acoustic and decorative. Processing of gypsum permits the use of different techniques to produce a range of these products: pressing for blocks, rolling, extrusion for panels and boards, casting for decorative elements and foam gypsum products.

25.2.3.1 Wall blocks

Ordinary and water-resistant gypsum binders, synthetic gypsum and normal-weight and lightweight concrete can be applied for manufacturing. Blocks can be solid or hollow. A common size is 390 × 190 × 188 mm. Compressive strength varies from 2.5

to 10 MPa. Specific energy consumption for 1000 bricks (250 x 120 x 65 mm) made of ordinary gypsum concrete is 12.4 kg of conventional fuel. For water-resistant gypsum concrete blocks, it is equal to 11.39 kg; for gypsum concrete with its low water requirement, it is 7.89 kg (Ferronskaya, 2004). According to Ferronskaya (2004), if equivalent power inputs on ordinary ceramic brick production will be accepted as 2504 kWh per 1000 of conventional brick as 100%, power inputs on the manufacturing of gypsum blocks are less than 1%.

When comparing the thickness of masonry walls of equal heat resistance but made of different materials, gypsum-based block walls usually have a lower thickness than ordinary clay or sand-lime brick. Even at the same average density gypsum-based walls have lower thickness than walls made of other materials like hollow clay bricks, claydite cement concrete (Fig. 25.16).

One suggested method for manufacturing walling blocks is based on phosphorgypsum processing, which permits excluding the grinding process of the binder (Fig. 25.17). It is an energy-efficient method for producing items directly from phosphorgypsum, providing the implementation of two chemical processes: dehydration of dihydrate and hydration of hemihydrate in one technological cycle. At ‘self-steaming’ in the high-density moulds, crystallised water releases in drip-liquid state and remains in the pores of grains and the voids of the gypsum. Blocks are pressed in moulds consisting of two punches and a formwork. The upper punch covers the gypsum powder

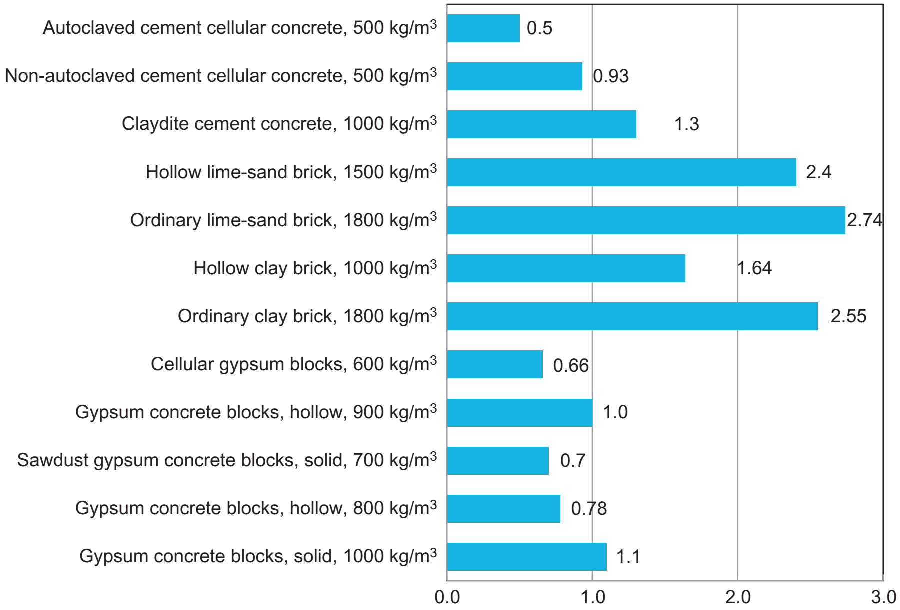

Fig. 25.16 Required thickness of conventional single-layer outwalls with equal thermal resistance (3.15 m^2 K/W).
Data from Dvorkin, L., Dvorkin, O., 2011. Building Mineral Binding Materials. Infra-Ingeneria, Moscow, pp. 27–90 (in Russian).

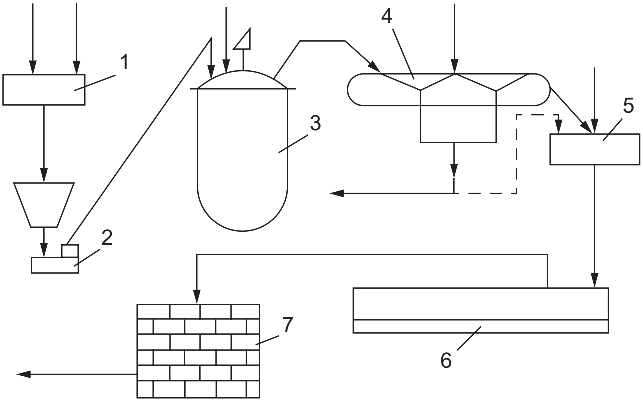

Fig. 25.17 Schematic circuit of phosphorgypsum direct-processing masonry units (grinding is excluded): Aggregate for phosphorgypsum modification (1); Pump (2); Continuously working autoclave (3); Filter (4); Mixing-activating equipment (5); Moulding machine (6); Masonry units (7).

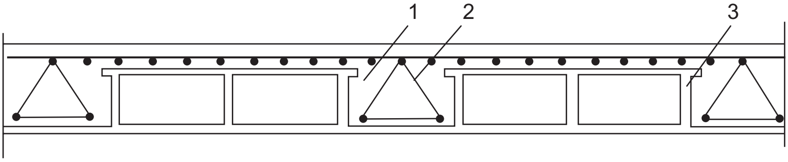

Fig. 25.18 GFRG panel. Cross section, schematic view: Cast-in-situ concrete (1); Reinforcing elements (2); GFRG panel (3).
Modified from GFRG[2], n/a. Glass Fiber Reinforced Gypsum Load Bearing Panels for Affordable Housing in Fast Track and Environmental Protection (Anon). Available at: http://www.frbl.co.in/rapidwall_for_housing.pdf.

placed in the formwork. With thermal treatment, the punch compacts the hydrated mass. The hardened product is demoulded at a temperature lower than 40°C.

Foam gypsum blocks can be produced with water-resistant binders or binders that are not water resistant. The blocks' density varies from 300 to 900 kg/m^3 and a compressive strength from 1.5 to 10 MPa. The most common size is 600 × 300 × 200 mm. They are applied for partitions and internal walls.

A new method of building construction based on GFRG panels was developed in Australia and is widely used in Asian countries (Fig. 25.18). It combines the benefits of prefabricated, lightweight large panels with cavities, conventional cast-in-situ concrete and steel reinforcement, which require specific design methods (GFRG$_2$, n/a; Wu, 2009).

Tongued and grooved panels are used for partitions (Fig. 25.19). Their length is 600–660 mm, their width is 300–500 mm and their thickness is 80–100 mm.

25.2.3.2 Gypsum boards and panels

Gypsum panel products are used for interior dry construction such as ceilings, walling and flooring. These elements are easily cut, soundproof and dimensionally stable.

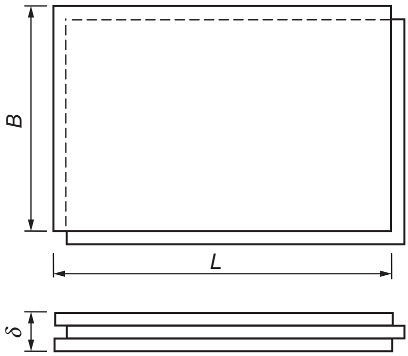

Fig. 25.19 Tongued and grooved gypsum panel. Schematic view.

Fig. 25.20 Cross section of plasterboard.

The most common element is drywall (plasterboard, veneer plaster), which is a gypsum plaster panel that is glued over two sheets of cardboard (Fig. 25.20). The most common plasterboard dimensions are 2500 × 1200 × 12.5 mm. The boards can be water-resistant and fire-resistant. The panels can be pre-decorated with water-resistant coatings (vinyl-faced, acryl-faced). In this case, they do not need additional finishing. Curved partitions and ceiling elements can be created for a finer aesthetic appearance.

Among gypsum products, it is mostly gypsum plasterboards that are subjected to regular recycling (Lund-Nielsen, 2008; Constructive demolition waste gypsum board recycling plant, n/a). Several technologies enable the use of recycled gypsum in the production of binders, for agricultural purposes and for the production of paper in the paper industry (Fig. 25.21).

Fibre-reinforced panels (fibreboards) are used extensively for partitions, ceilings and floor underlayments (Fig. 25.22). Walling and ceiling plasterboards and fibreboards are installed on steel or wooden frameworks. Panels and boards can also be glued to prepared, smooth surfaces without a framework.

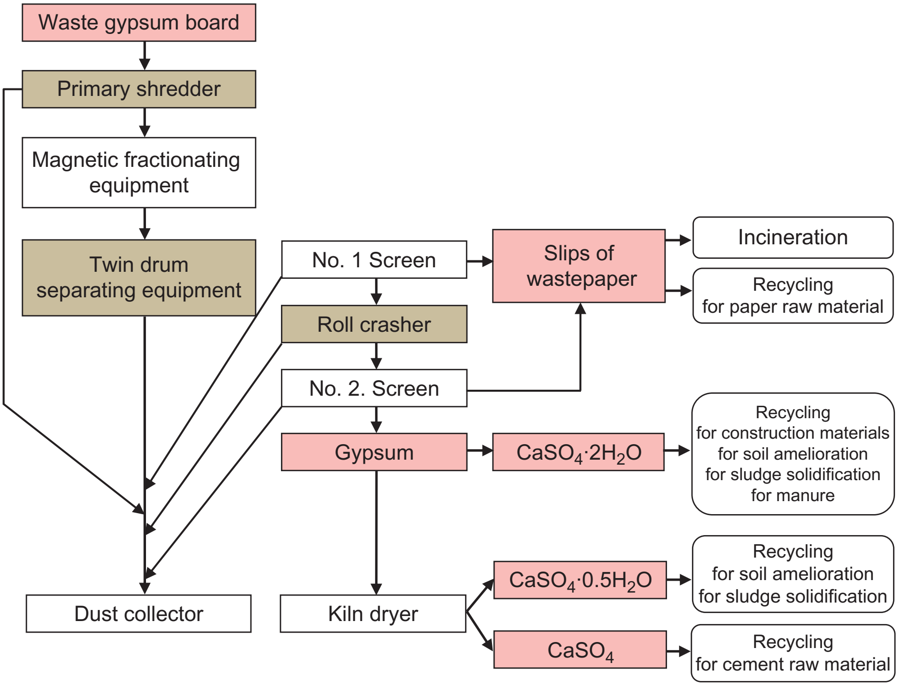

Fig. 25.21 Flow diagram of recycling of gypsum board wastes.
From Constructive demolition waste gypsum board recycling plant, n/a. Available at: http://www. fujikogyo.co.jp/HP-English/sekko/sekko.html.

Fig. 25.22 Flooring gypsum fibreboards, thickness 12.5 mm.

Acoustical panels are made of fibre-reinforced gypsum or with an underlaying of mineral wool. They have different perforations (Fig. 25.23A). Perforated gypsum board panels can also be used for acoustic ceilings (Fig. 25.23B). Acoustical tile cross sizes are normally 600 × 600 and 800 × 800 mm. Thickness is 30 or 40 mm. Perforation area is about 10–15%.

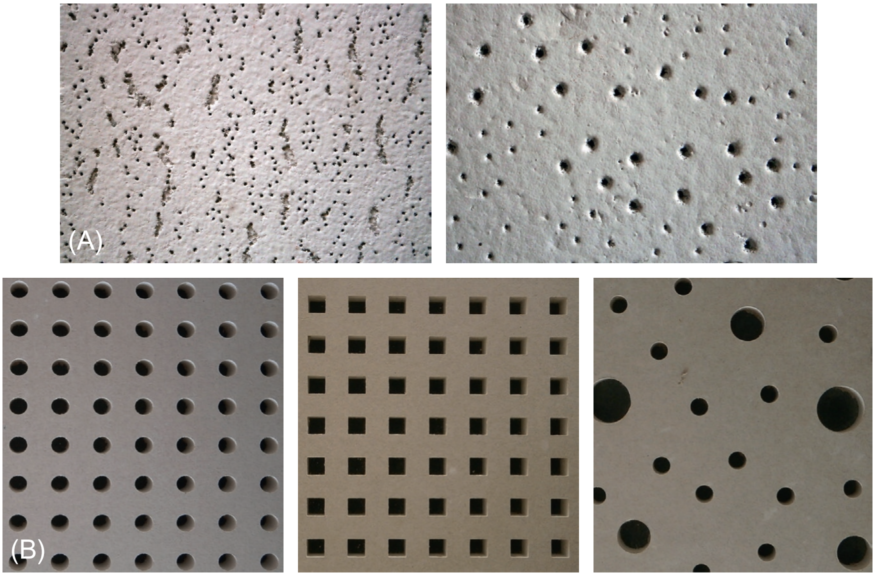

Fig. 25.23 Textures of acoustic panels: (A) Fibre-reinforced gypsum with underlaying of mineral wool; (B) Gypsum board perforated panels.

25.2.3.3 Decorative elements

Decorative panels are produced with high-strength building gypsum and polymer-modified gypsum. They are used for interior decoration. The typical length and width are 600 mm, and thickness is between 15 and 25 mm. *Stuccoworks* (rosette mouldings, cornices and other decorative details) are produced by casting in gypsum, concrete, metal, wooden or plastic moulds (Fig. 25.24). Mostly high-strength gypsum of high whiteness is applied. Lightweight details are also used for interior facings (Fig. 25.25).

Gypsum plasters are used for interior finishing. Prefabricated mixtures of dry components are normally ‘dry packed’ as one component. They are mixed with water at the construction site immediately before application. According to their use, gypsum plasters can be divided, for example, into plastering (undercoat or finishing), spackling and flooring (self-levering).

Plastering mixtures usually consist of non-water-resistant β and α-hemihydrates, water-resistant (gypsum-cement-pozzolanic) binders, fine-grained aggregates less than 2.5 mm in size, special chemical admixtures and mineral additions. They are used to smooth different surfaces: brickwork and normal-weight and lightweight concrete. *Spackling plasters* contain fine-grained fillers and are used for filling holes and for smoothing surface defects. Because of the rapid hardening of gypsum, spackling plasters permit finishing work to begin after only a few hours of hardening. *Flooring (self-levering) plasters* include α-hemihydrates, anhydrite, Estrich gypsum, water-resistant binders and water-reducing and water retaining

Fig. 25.24 Stuccowork, replicating decorative pediment of the façade of Saint Sophia's Cathedral, Kiev, Ukraine.

Fig. 25.25 Interior facing element made of pigmented gypsum.

admixtures. *Masonry plasters* are used for masonry work on partition panels, on basement floors (non-water-resistant binders) and on walling blocks (water-resistant binders).

In renovation, gypsum-lime-sand mortars are normally used (Reichel et al., 2005). The plasters may also contain lime, vermiculite and mineral and chemical admixtures. Because gypsum hardening is neutral, most admixtures are not as efficient as cement is. Fibre-reinforced foam gypsum slabs and foam polymer gypsum are heat-insulating products (Ferronskaya, 2004).

25.2.3.4 Other products

Gypsum (mostly synthetic ones) is also used to retard the setting time of admixtures for cements. The application of synthetic gypsum may lead to a decrease in the strength of parameters. Research conducted by Khatib et al. proved that partial replacement of cement for FGD gypsum increases the threshold diameter of the pores in cement pastes (Khatib et al., 2013, in press-a) and a reduction in the strength of concrete (Khatib et al., in press-b).

Because of its high rate of hardening and an initial swelling with subsequent shrinkage, high-purity gypsum is used as moulds in the pottery industry. Used moulds can be recycled for recurring dehydration and grinding.

Gypsum is also used in glass manufacturing, for high-strength floor underlays, as a patching material for road and surface repairs and for thermoplastics, thermosets and coatings, as well as for other construction industries (Ghrefa and Howari, 2011).

25.3 Sustainability aspects

Defined green features of sustainable construction materials can occur at all stages of the materials' life cycle, from the point of being a raw material to the point of disposal of its wastes (Kim and Rigdon, 1998). Sustainable materials are normally produced from natural, local or recycled materials. The manufacturing process can reduce waste and prevent pollution because of recycling and advanced technologies. During construction and its performance, a product should be nontoxic and durable and should provide an energy-saving building process and result. At the end of its life, in terms of waste management, a green, sustainable material needs to be either biodegradable, or recyclable or reusable. The total energy consumption 'accumulated' by a product during all stages of its life cycle, called 'embodied energy', should be as low as possible.

25.3.1 Embodied energy and carbon footprint of gypsum products

Embodied energy is the accumulative energy spent for a product's total life cycle, from raw mining to disposal, and it is considered to be incorporated in the material itself. The estimation of a construction material's from cradle-to-gate embodied energy is an estimation of the energy spent from the time of its extraction until the time it reaches the factory gate. Because of the wide range of manufacturing methods and the complexity of calculations, transportation distances and other variables for some building products, values of embodied energy vary from study to study (BEDB, 2009).

Carbon footprint value, which is estimated along with embodied energy, relates to the accumulated greenhouse gases caused by a product during its life cycle. According to research results presented by the University of Bath (United Kingdom), the embodied energy of gypsum plaster is about 1.8 MJ/kg, and its carbon footprint is 0.12 kg CO_2 per 1 kg of product. This could be compatible with terrazzo tiles with corresponding values of 1.4 MJ/kg and 0.12 kg CO_2 per 1 kg of product. In addition, gypsum plasterboard has an embodied energy of 6.75 MJ/kg and 0.38 kg CO_2 per 1 kg of product, which is comparable to clay tile values of 6.5 MJ/kg and 0.45 kg CO_2 per 1 kg of product (Hammond and Jones, 2006).

The embodied energy and carbon footprint of gypsum products for walling assemblies in the United States along with other relevant materials according to the data of the Buildings Energy Data Book (BEDB, 2009) are shown in Fig. 25.26. In general, a 60-year building life time was estimated. As can be seen, gypsum board partitions with wood studs have the lowest values of these parameters when compared to the other products.

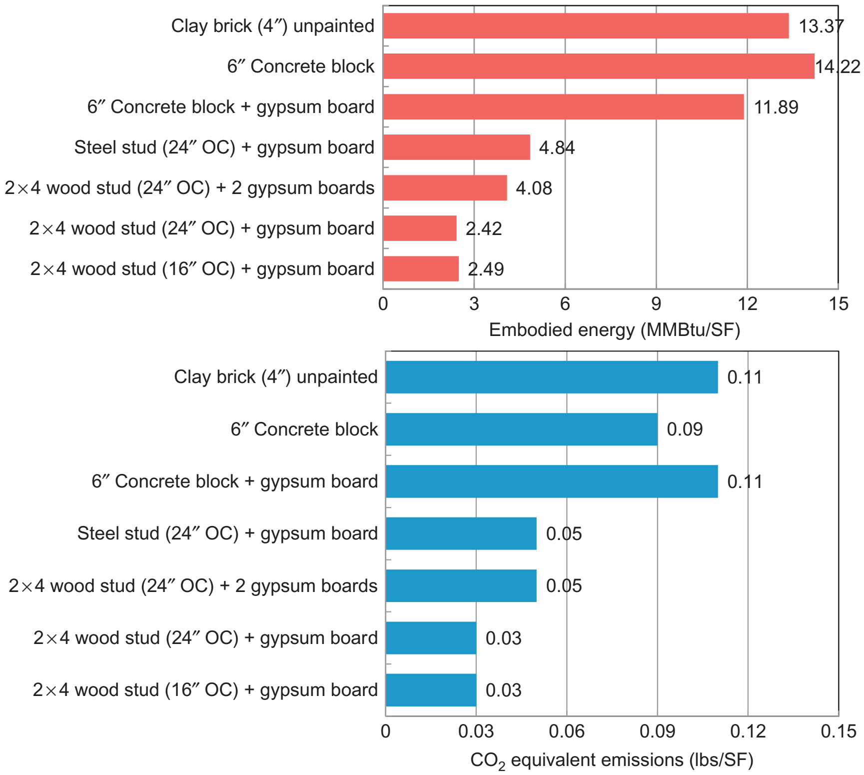

Fig. 25.26 Embodied energy and CO_2 for interior wall assemblies in the United States. 1 MMBtu = 1055 MJ, 1 sq ft = 0.093 sq m. 1 lbs = 0.454 kg.
Based on data from BEDB, 2009. Building Energy Data Book, 1.6.6 Embodied Energy of Interior Wall Assemblies in the U.S., pp. 1–37. Prepared for the Buildings Technologies Program. Energy Efficiency and Renewable Energy U.S. Department of Energy by D&R International, Ltd., 1.6.6 Embodied Energy of Interior Wall Assemblies in the U.S. Available at: http://static1.squarespace.com/static/513f072ae4b0a96a24469023/t/5410b01ae4b05f0d2fb6861d/1410379802995/docs_DataBooks_2009_BEDB_Updated.pdf, which is based on calculations of Athena EcoCalculator for Assemblies version 2.3, 2007.

25.3.2 Reusing and recycling gypsum products

Direct reuse of gypsum products after demolition is unusual. Thus careful demolition of plasterboards and panels is needed in order to reuse them. Normally, recycling procedures are used for gypsum products, mostly plasterboards and wastes of gypsum manufacturing.

Disposal of gypsum products has become a serious issue because they still contain various substances such as organic matter (paper and wood), drywall (made mainly of lining paper and gypsum $CaSO_4{\cdot}2H_2O$) and heavy metals. The wastes are regarded as the major contributor to hydrogen sulphide (H_2S) generation in landfills (Musson et al., 2007). Wastes formed by the production of gypsum products can be recycled instantly at the point of processing. Lump wastes of gypsum concrete or gypsum plasters at large enterprises can be ground and returned to the manufacturing process. Gypsum dust accumulated in dust-collecting systems can be used as a setting regulator in the mixtures or can be added to gypsum binder during homogenisation (Ferronskaya, 2004).

Construction and demolition wastes require preliminary treatment (removal of impurities, etc.). There are developed technologies for distribution of gypsum and organic matters in demolition and construction wastes. Heavy liquid separation can be applied to determine the density range in the places of the highest concentration of gypsum and organic wastes. However, as heavy liquid separation requires water and hazardous chemicals to be applied, is more advantageous (Montero et al., 2010).

Different kinds of equipment are used for waste processing, such as wasteboard crushers, which are developed especially for recycling rejected gypsum boards and their wet and dry wastes (Gypsum Technik, 2014). Gypsum Recycling International, a mother company for the operating units in the Gypsum Recycling Group, has developed mobile gypsum recycling units. Trailer-based and transported by ordinary trucks, these units can recycle 100,000 tonnes per year. The units recycle both new gypsum waste and waste from reconstruction and demolition (Lund-Nielsen, 2007). According to calculations of Gypsum Recycling International, 200 kg of CO_2 is saved for each recycled tonne of waste. One hundred percent of paper recycling is achieved for gypsum paper recycling. Developed in Denmark in 2001, such recycling activity has spread to other European countries, North America and Asia (Lund-Nielsen, 2007).

Other recycling technologies exist, and they deal mostly with demolition wastes, recycled at plasterboard manufacturing lines. Yoshino Gypsum Co., Ltd. (Japan) uses by-product gypsum (50%), imported raw gypsum (45%) and collected waste plasterboards (5%) for domestic plasterboard production; 100% of recycled paper is applied for production (Environmentally-Friendly Gypsum Products, n/a). The Australian company ReGyp Pty Ltd suggests solutions not only for manufacturing construction products but also for agriculture. Ninety-five percent of recycled gypsum dehydrate are applied soil addings (http://www.regyp.com.au). The level of application of recycled and synthetic gypsum varies from country to country. In the United States in 2014, more than four million tonnes of gypsum scrap generated by wallboard manufacturing, wallboard installation and building demolition were recycled. The recycled gypsum was used primarily for agricultural purposes and as feedstock for the manufacture of new wallboard. Other potential markets for recycled gypsum include athletic field marking, cement production as a stucco additive, grease absorption, sludge drying and water treatment. In 2014 in the United States, synthetic gypsum accounted for approximately 50% of the total domestic gypsum supply (MCS, 2015). In the United Kingdom, 38% of imported gypsum stone, 51.6% of FGD gypsum (one-third of it imported), 5.6% of titanogypsum and 4.8% of recycled gypsum are used for plasterboard production (ELCAP, 2010).

Use of gypsum wastes and synthetic gypsum is vital for the countries where natural gypsum deposits are rather rare (Japan, South Korea, New Zealand and Australia, etc.). Share of FGD gypsum in European Union plasterboard production has reached 60% in 2010, and it was close to zero in the middle of 1970s (Lund-Nielsen, 2010). The increasing demand to meet the standards of sustainable construction (LEED in the

United States, BREAM in the United Kingdom, etc.) is promoting ‘green’ technologies related to the manufacturing, use and disposal of gypsum products.

According to Lund-Nielsen (2008) the reasons plasterboard enterprises want to become involved in recycling schemes vary depending on the region (Table 25.6). As can be concluded, the limited resources of natural gypsum as well as the higher cost of disposing and transporting it are dominant factors.

25.3.3 Life-cycle assessment

Life-cycle assessments (LCAs) of gypsum products are generally considered to be from cradle-to-gate. The use and disposal stages are excluded from the assessments. Numerous data have been collected for different gypsum products, but mostly for wallboard manufacture. The LCA of the most commonly recycled gypsum product, gypsum board, is based on case-study research conducted by the order of the International Gypsum Association (EPD NAGB, 2014; LCAS, 2013). LCAs have been developed according to ISO 14025 and 21930 principles and requirements.

Calculations were conducted for 1000 sq ft (1007 kg) of 5/8″ type X gypsum board at the facility’s gate. The calculation included 2117.66 lbs (960.52 kg) of gypsum material consisting of natural gypsum ore (40.5%), FGD synthetic gypsum (57.5%) and post-consumer gypsum (2%). Gypsum paper (both facing and backing) consumption was 84.6 lbs (38.37 kg). Additives were used: 1348.6 lbs (611.71 kg) of starch, vermiculite, fibreglass, dispersant, retarder, potassium sulphate, dextrose, clay (kaolin), boric acid, land plaster, foaming agent (soap), BM accelerator, ammonium sulphate, edge paste, shredded paper and water. The ‘Cradle-to-Gate’ Gypsum Board Manufacturing System is shown in Fig. 25.27. Basic LCA results are shown in Table 25.7.

Fig. 25.28 shows the relative contributions from the raw material input supply, input transport to a manufacturing facility and gypsum board manufacturing, showing their dominance in the impact categories. Gypsum board manufacturing has a much greater influence on the environment than do raw supply and transportation, particularly on global warming, acidification, ozone depletion and abiotic depletion potentials.

The Environmental Product declaration prepared by GIPS Bundesverband der Gipsindustrie e.V. (Germany) suggests a comparative LCA analysis of different gypsum-based products (GBP EPD, 2009). The LCA was conducted in compliance with DIN ISO 14040. Primary energy consumption is shown in Fig. 25.29. For each product, the nonrenewable energy is more than 90% of the total energy consumption.

According to this document, the highest values of global warming potential and acidification potential among all ready-to-use products result from gypsum fibreboards (0.308 kg CO_2 eq./kg and 0.39E−03 SO_2 eq./kg correspondingly) and fibreboard flooring elements (0.303 kg CO_2 eq./kg and 0.38E−03 SO_2 eq./kg correspondingly). The highest values of eutrophication potential are peculiar to all of the plasterboards: for perforated plasterboard, 8.21E−05 kg PO_4 eq./kg; for fire-retardant plasterboard, 8.05E−05 kg PO_4 eq./kg; for precast screed plasterboard, 7.66E−05 kg PO_4 eq./kg.

The products with the greatest potential for creating ozone are gypsum perforated (4.01E−05 kg C_2H_4 eq./kg) and fire-retardant plasterboard (3.87E−05 kg C_2H_4 eq./kg), fibreboards for walling (3.7E−05 kg C_2H_4 eq./kg) and flooring (3.76E−05 kg C_2H_4 eq./kg).

Table 25.6 **Factors influencing plasterboard plants to become involved in recycling (the major drivers are in bold)**

Main drivers	**Europe**	**US and Canada**	**Japan and Korea**	**Australia and New Zealand**	**Middle East**
Legislation	**EU landfill directives increase disposal cost** **Landfill taxes** **Recycling targets for C&D waste** **Extended producer responsibility**	**Ban on landfilling Of waste (Canada)** **Ban on gypsum in daily landfill cover materials**	**Special rules for landfilling of gypsum waste**	Landfill taxes	Small influence
Scarce supply of inexpensive raw materials	Lack of cheap sources nearby (for Nordic countries) Shrinking supply of FGD/DSG (in northern continental Europe)	Higher cost of transporting alternative gypsum raw materials (northern US)	Lack of cheap sources nearby Higher cost transporting alternative gypsum raw materials	Lack of cheap sources nearby Higher cost transporting alternative gypsum raw materials	Lack of cheap sources nearby Higher cost transporting alternative gypsum raw materials
Customer pressure	**Increasing cost of disposal, general recycling mentality and a fear of global warming** **Industry agreements (UK, France, NL) to commit to help and develop recycling**	**The LEED system**	**High cost of disposal and fear of global warming**	**Rising cost of disposal and fear of global warming**	US companies want LEED projects

Data from Lund-Nielsen, H., 2008. Recycling of plasterboard waste—from ‘nice to have’ to a ‘necessity’. Global Gypsum, September, pp. 30–40.

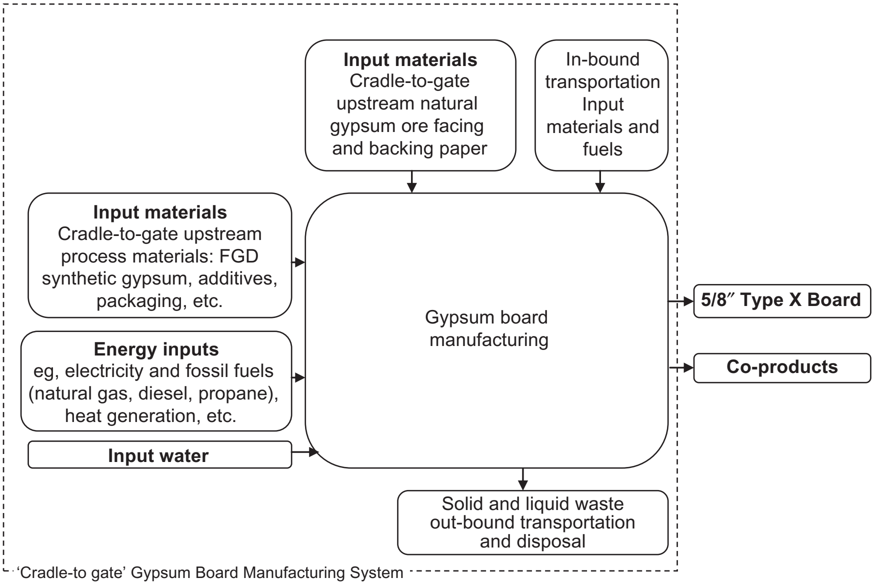

Fig. 25.27 System boundary and process flows for gypsum board manufacturing. Data from EPD NAGB, 2014. Environmental Product Declaration. Typical (5/8″ Type X) North American Gypsum Boards. Declaration number: FPI/GA/01/2014 Issued May 2014 Valid until May 2019. Available at: http://www.gypsum.org/wp/wp-content/uploads/2013/12/Gypsum-2014-FINAL-May-13-.pdf.

Table 25.7 LCA results of environmental impact per 1000 sq ft of gypsum board, with 5/8″ thickness

Parameter	Units	Total of the product stage
Global warming potential	kg CO_2 eq.	317.4
Depletion potential of the stratospheric ozone layer	kg CFC-11 eq.	1.9E−05
Acidification potential	kg SO_2 eq.	2.46
Eutrophication potential	kg N eq.	0.37
Smog potential	kg O_3 eq.	12.41
Abiotic depletion potential of fossil fuels	MJ	596.1

Data from EPD NAGB, 2014. Environmental Product Declaration. Typical (5/8″ Type X) North American Gypsum Boards. Declaration number: FPI/GA/01/2014 Issued May 2014 Valid until May 2019. Available at: http://www.gypsum.org/wp/wp-content/uploads/2013/12/Gypsum-2014-FINAL-May-13-.pdf.

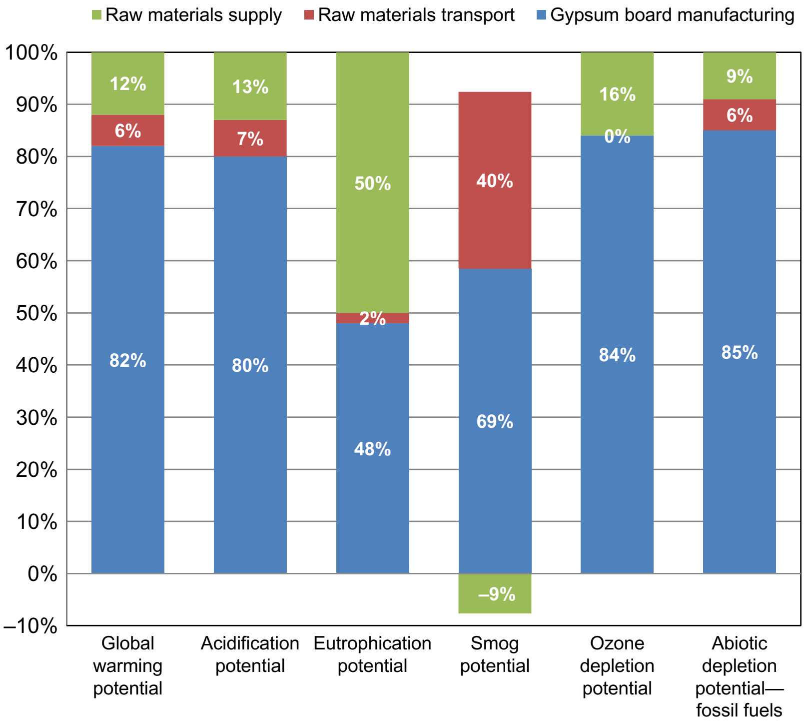

Fig. 25.28 Net impact of FGD synthetic gypsum use resulted in a net benefit to the environment because of its diversion from landfilling.
Data from EPD NAGB, 2014. Environmental Product Declaration. Typical (5/8″ Type X) North American Gypsum Boards. Declaration number: FPI/GA/01/2014 Issued May 2014 Valid until May 2019. Available at: http://www.gypsum.org/wp/wp-content/uploads/2013/12/Gypsum-2014-FINAL-May-13-.pdf.

Of all the considered parameters, the products with the lowest potential are those with supplementary jointing gypsum materials and adhesive binders. Therefore, the potential for global warming is equal to 0.108 kg CO_2 eq./kg; for photochemical ozone depletion, the potential is 1.33E−05 kg eth4 eq./kg, and a acidification potential of 0.14 SO_2 eq./kg; and for the eutrophication, the potential is 1.55E−05 kg PO_4 eq./kg (GBP EPD, 2009).

25.4 Future trends

The available data indicate a slight increase in gypsum mining. China will be the world's largest producer of natural gypsum for the next few years. However, because of the depletion of deposits of natural gypsum and anhydrite, the use of synthetic gypsum and recycled gypsum will increase. As there are still a significant number of synthetic gypsum landfills, manufacturers will appear to recycle it.

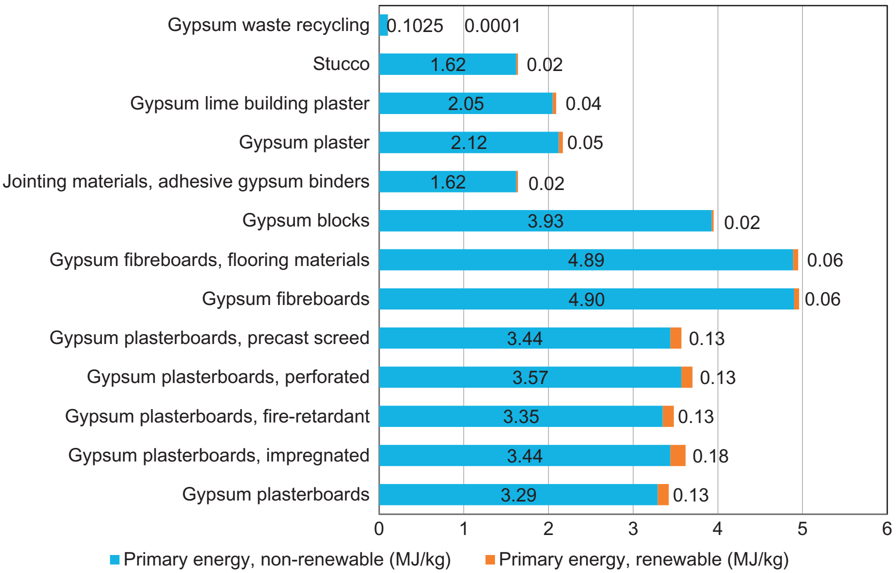

Fig. 25.29 Primary energy consumption for gypsum products.
Data from Forschungsvereinigung der Gipsindustrie e. V. GBP EPD, 2009. Gypsum-based products Environmental product declaration, first ed. Bundesverband der Gipsindustrie e.V. Forschungsvereinigung der Gipsindustrie e.V., 16 pp. Available at: http://www.hunton.no/wp-content/uploads/2013/06/EPD-Gypsum-Fibreboards-EN15283-2-Produktdeklaration_E_09-06-09.pdf.

A Roskill report forecasts that by 2018 FGD gypsum could contribute more of the world's supply of gypsum than natural gypsum does. Overall, the gypsum market is expected to increase by 5% until 2018 (Gypsum and Anhydrite: Global Industry Markets and Outlook, 2014). For the European plasterboard industry, Lund-Nielsen (2010) predicts a 25% increase in the use of recycled gypsum over the next 20 years (Fig. 25.30).

It is predicted that over the next 15–20 years, we will 'see the first new plants that are located close to the end-markets and being built to use 100% recycled gypsum' (Lund-Nielsen, 2010).

25.5 Sources of further information

25.5.1 Information about the gypsum industry

Information about gypsum products can be found in following books: *Gypsum: Properties, Production, Application* (Sampson, 2011) and *Gypsum Construction Handbook* (Greene, 2009); see also international journals such as *Construction and Building Materials* and the online journal *Global Gypsum*.

Official websites of gypsum associations and producers are also useful, including the not-for-profit trade association founded by gypsum industries from the United

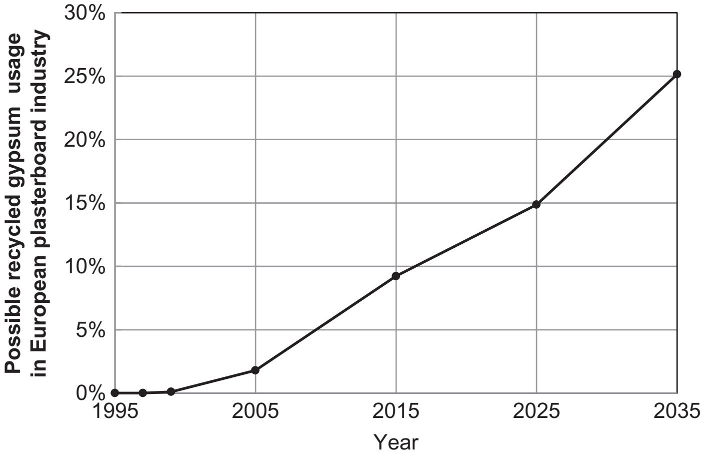

Fig. 25.30 Possible evolution of the use of recycled gypsum in the European plasterboard industry.
Data from Lund-Nielsen, H., 2010. European long-term trends in recycled gypsum usage. Global Gypsum Magazine, November, pp. 24–28.

States and Canada (http://www.gypsum.org) and the Gypsum Products Development Association (GPDA), which represents the four major gypsum board and plaster manufacturers in the United Kingdom and Ireland—British Gypsum, Lafarge Plasterboard, Knauf Drywall and Gypsum Industries (http://www.gpda.com). National Gypsum (United States) is one of the largest gypsum board producers in the world (http://www.nationalgypsum.com); British Gypsum is the United Kingdom's leading manufacturer of interior lining systems (http://www.british-gypsum.com). The Association of the European manufacturers of gypsum products (Eurogypsum) represents the gypsum industry in Europe (http://www.eurogypsum.org). Some of the largest global manufacturers of gypsum products include Saint Gobain, France; Gyproc group (http://www.gyproc.com/); Lafarge, France (http://www.lafarge.com/en/gypsum); Knauf, Germany (http://www.knauf.com); USG Corporation, United States (http://www.usg.com); and Yoshino Gypsum Co. Ltd, Japan (http://www.yoshino-gypsum.com).

25.5.2 Information about sustainable resources management

Information on sustainable resource management of gypsum products for construction can be found on manufactures' websites.

Information about environmentally friendly gypsum products and gypsum product recycling techniques can be found at Gypsum Recycling International's website, the mother company of the operating units in the Gypsum Recycling Group, Denmark (http://gypsumrecycling.biz/); and at the website for the European collaborative project between the recycling industry, the demolition sector and the gypsum industry (http://gypsumtogypsum.org). Recycling solutions for the Australian gypsum industry can be found at http://www.regyp.com.au, and technical and environmental information about the use of FGD gypsum can be found at http://www.fgdproducts.org.

Information about the outlook for gypsum and anhydrite global can be found in annual reports by Roskill (Gypsum and Anhydrite: Global Industry Markets and Outlook, 11th edition 2014, http://www.roskill.com/reports/industrial-minerals/gypsum).

Appendix Standards for gypsum products

ASTM standards

C22/C22M	Specification for gypsum
C28/C28M	Specification for gypsum plasters
C35	Specification for inorganic aggregates for use in gypsum plaster
C36/C36M	Specification for gypsum wallboard
C37/C37M	Specification for gypsum lath
C52	Specification for gypsum partition tile or block
C59/C59M	Specification for gypsum casting plaster and gypsum moulding plaster
C61/C61M	Specification for gypsum Keenes cement
C317/C317M	Specification for gypsum concrete
C318/C318M	Specification for gypsum formboard
C472	Test methods for physical testing of gypsum, gypsum plasters and gypsum concrete
C473	Test methods for physical testing of gypsum panel products
C475/C475M	Specification for joint compound and joint tape for finishing gypsum board
C557	Specification for adhesives for fastening gypsum wallboard to wood framing
C587	Specification for gypsum veneer plaster
C588/C588M	Specification for gypsum base for veneer plasters
C843	Specification for application of gypsum veneer plaster
C1396/C1396M	Specification for gypsum board

BS standards

BS EN 520:2004+A1:2009	Gypsum plasterboards. Definitions, requirements and test methods
BS EN 12859:2011	Gypsum blocks. Definitions, requirements and test methods
BS EN 13279-1:2008	Gypsum binders and gypsum plasters. Definitions and requirements
BS EN 13279-2:2014	Gypsum binders and gypsum plasters. Test methods
BS EN 13815:2006	Fibrous gypsum plaster casts. Definitions, requirements and test methods
BS EN 13950:2014	Gypsum board thermal/acoustic insulation composite panels. Definitions, requirements and test methods
BS EN 13963:2014	Jointing materials for gypsum boards. Definitions, requirements and test methods
BS EN 14190:2014	Gypsum board products from reprocessing. Definitions, requirements and test methods
BS EN 15283-2:2008+A1:2009	Gypsum boards with fibrous reinforcement. Definitions, requirements and test methods. Gypsum fibre boards.

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Sustainability of desulphurised (FGD) waste in construction

26

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26.1 Introduction

Over recent decades, there has been increasing pressure on governments to consider the effects of industry on the environment. An example of this is the UK Environment Agency (UK EA, 2015) that deals with environmental issues within England and Wales, which works in close relation with other organisations such as the European Environmental Agency (EEA, 2015). With the development of these agencies, along with other environmental organisations, increasing pressure is being placed on industry to improve its impact on the environment. Historically, the power industry has been a major cause of pollution through the burning of fossil fuels. With increasing awareness over the past few decades, the power industry has been forced into developing methods to produce more environmentally friendly ways of producing power, such as wind, solar, and tidal. Until it is possible to implement these methods on a larger scale that is both efficient and cost effective, it is necessary to make the systems currently being used as environmentally friendly as possible.

The current consensus is that the use of raw materials such as black and brown coal in the production of power will continue worldwide. In reality, global consumption of coal has been continuously increasing (Khatib, 2014), especially in light of recent issues surrounding the implementation of nuclear power. On combustion, these fossil fuels emit SO_2, NO_x, and CO_2, which play a major part in the pollution of the environment. One such problem associated with these emissions is acid rain, which can restrict the growth of plants, pollute freshwater areas, and even contribute to the deterioration of buildings. Many countries have implemented standards and legislation to combat the effects of air pollution and effects on the environment. One such control is the reduction of sulphur dioxide (SO_2) from power stations (Smith, 2006).

The reduction of SO_2 emissions produced by power installations can be achieved by installing desulphurisation systems to new and existing stations. These systems generally introduce alkaline sorbents such as limestone to the SO_2 gasses, which react and form insoluble wastes such as gypsum ($CaSO_4 \cdot 2H_2O$).

In 1993, it was estimated that the number of flue-gas desulphurisation (FGD) installations fitted to power stations throughout the world exceeded 600 on a total capacity of 160 GWe, with a further 100 installations planned at that time. It was estimated that by 2000, the amount of desulphurised wastes produced each year would exceed 60 Mt (Clarke, 1993). The IEA Clean Coal Centre database had over 900 FGD installations on record in 2005 with a total capacity of 316 GWe and with further increases planned (Smith, 2006).

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The efficiency of desulphurisation systems with respect to reducing SO_2 emissions varies significantly, depending on the type of facility and process of desulphurisation installed, and therefore the wastes produced also vary with respect to chemical and mineralogical composition.

Due to the variability of materials produced from different desulphurisation processes, and impurities present in many residues produced, many wastes have rarely been utilised, and have been disposed of in landfill sites. In recent times, this disposal to landfill sites is becoming problematic from an environmental perspective because of a reduction in available sites and increasing disposal costs. There is also greater awareness of the effects that landfill wastes may have on the environment through leaching of elements into the soil and groundwater (Clarke, 1993).

Currently, materials from FGD systems are used in the construction industry, especially wet FGD wastes, which are used in the manufacture of plasterboard, gypsum blocks, floor screeds, and plasters, and in cement production. Other FGD wastes have been used in the production of recycled lightweight aggregates, which can be used in concrete or masonry blocks. Other applications include manufacture of foamed concrete, flowable fills, however, substantial amounts of FGD are still sent to landfills.

The key to the utilisation of wastes within the construction industry is characterising and classifying different wastes, and developing their use in high-volume applications, which can be manufactured at source to reduce production and transport costs. This would also reduce the need for natural materials to be mined, which in turn would reduce the impact on the environment from both the power and construction industries.

26.2 Desulphurisation processes

There are many different desulphurisation processes available at present that produce a variety of wastes that vary in quality and quantity (Smith, 2006; Clarke, 1992). The type of waste is dependent on many factors, including process and raw materials used. The majority of processes fit into three main categories, wet, semidry, and dry processes. However, other processes are available, which are based on, or a combination of, the three methods mentioned, such as circulating fluidised bed (CFB), gas suspension and absorption (GSA), and fluidised bed combustion (FBC).

The wet process is at present the most environmentally friendly in terms of purity and quantity of materials produced for exploitation. Desulphurisation processes use an alkaline sorbent such as limestone, slaked lime, or a mixture of slaked lime and fly ash, which is mixed into slurry and sprayed into the flue gasses. The most common method uses limestone as a sorbent, which reacts with the SO_2 gasses to form an insoluble calcium compound, which is then treated to produce calcium sulphate ($CaSO_4{\cdot}2H_2O$-gypsum) (Clarke, 1993). The gypsum purity of the wastes from wet processes normally exceeds 96%. This makes the wastes produced by the wet process suitable for use in the construction industry in areas such as wallboard manufacture and cement production. Wet processes are expensive to implement, run, and maintain, however, work has been carried out to improve the cost efficiency of such processes (Achenbach, 1993; Klingspor et al., 1998). Even though installations are becoming more cost efficient, historically many smaller countries did not have the financial resources to install such processes, so more economically viable options were installed, such as dry and semidry processes (Clarke, 1993).

The semidry process (spray dry scrubbers) introduces an alkaline solution or slurry to the flue gasses in the reaction vessel. Once the reaction occurs, the solution is evaporated, salts are precipitated, and the remaining solids are dried. Although the wastes are dried, the moisture content can still be as high as 2%, which is why they are commonly referred to as semidry wastes. The general difference between the semidry and wet process is that alkaline sorbent (slaked lime) and fly ash are entrained into the flue gasses. This makes the final wastes a mixture of calcium sulphite hemihydrate, calcium sulphate dihydrate, and fly ash. In some installations, the fly ash is sometimes separated from the calcium sulphate/sulphite wastes. Therefore, the composition of wastes from semidry processes can vary significantly (Clarke, 1993). Given that semidry wastes are a mixture of calcium sulphate, sulphite, and fly ash, they are not commonly used in construction applications.

The dry method (sorbent injection) introduces the alkaline sorbent in dry form to either/or a combination of, the fuel source, the furnace, or the ductwork. As with the semidry method, the waste produced is a combination of reacted and nonreacted sorbent along with fly ash. These types of systems are more likely to be fitted to older power stations, and their future use is limited due to their poor SO_2 removal (50–70%) and the type and quantity of wastes produced.

26.3 Chemical composition, evaluation, and classification

The following chapter introduces the different types of desulphurised wastes currently being produced from wet, semidry, and dry FGD processes, with focus on the variation in chemical and mineralogical composition and the methods used to evaluate and classify these wastes.

26.3.1 Chemical composition

Wastes from wet processes are mainly calcium sulphate/sulphite based. The most common waste formed is gypsum ($CaSO_4\cdot 2H_2O$) with other forms, such as anhydrite ($CaSO_4$), hemihydrate ($CaSO_4\cdot|H_2O$, plaster), and calcium sulphite ($CaSO_3\cdot|H_2O$), also produced. In the UK, gypsum is produced to a desired purity mainly for use in the wallboard industry. Tables 26.1 and 26.2 show the chemical composition and major oxide content of typical UK gypsum (Smith, 2006).

Desulphurised (or FGD) waste materials from dry and semidry processes, or a combination of the two, contain a mixture of fly ash and calcium sulphate. These types of wastes have been referred to as SiO_2-Al_2O_3-$CaSO_4$-based desulphurised wastes (Mangat, 1999; Mangat et al., 2006). Table 26.3 shows a selection of SiO_2-Al_2O_3-$CaSO_4$-based

Table 26.1 Typical chemical composition of gypsum

Property									
Free moisture	**Purity ($CaSO_4 \cdot 2H_2O$)**	**$CaCO_3$**	**$CaSO_3$**	**Chloride**	**K + M + N**[a]	**SiO_2**	**Fe_2O_3**	**pH**	**Crystal size**
≤10%	≥95%	≤5%	≤0.5%	≤100 ppm	≤0.1%	≤1.5%	≤0.04%	6–8	75% >16 μm

[a] $K_2O + MgO + Na_2O$—with $Na_2O \leq 0.05\%$, ≤1.0% as respirable.

Table 26.2 **Typical major oxide content of gypsum**

Major oxide (% weight)							
SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	SO_3	K_2O	Na_2O
0.38	0.21	0.07	35.55	0.02	49.20	0.02	0.01

Table 26.3 **Desulphurised wastes investigated during the Copernicus project (Mangat, 1999)**

Country	Name of resource	Type of resource	Process	Reference
Poland	RYBNIK	EPS	Semidry/Dry (LIFAC)[a]	PL-RP
			Dry (FGD)[b]	PL-RS
	PUSTKOW	Chemical factory	Semidry[b]	PL-P
	ZERAN	TPS	CFB[a]	PL-ZP
			CFB[a]	PL-ZZ
	LAZISKA	EPS	Semidry (NID)[b]	PL-L
Czech Republic	ZLIN	TPS	FBC (Bed)[a]	CZ-ZB
			FBC (Cyclone)[a]	CZ-ZC
			FBC (Filters)[a]	CZ-ZF
	SKODA	TPS	Semidry[b]	CZ-SFA
			Semidry[b]	CZ-SD
Slovak Republic	NOVAKY	TPS	Dry FBC (Flue)[a]	SL-F
			Dry FBC (Bed)[a]	SL-B
		TPS	Wet[b]	SL-FA
			Wet[b]	SL-G
Holland	ROCAL	DP	Wet[b]	ANH-RO
	Dordrecht	EPS	FBC[a]	FBC-B
Germany	Duisburg	EPS	FBC[a]	FBC-A
Denmark	IS Midkraft	EPS	SDA[b]	SDA

Bed, cyclone, filters, flue, location of particulate collection unit; *CFB*, circulation fluidised bed; *DP*, demonstration plant for converting FGD gypsum into anhydrite; *EPS*, electricity power station; *FBC*, fluidised bed combustion; *FGD*, flue-gas desulphurisation; *LIFAC*, limestone injection into the furnace and reactivation of calcium; *NID*, new integrated desulphurisation; *SDA*, spray dry absorption; *TPS*, thermal power station.

[a] Desulphurisation occurs during combustion of fuels.

[b] Desulphurisation occurs after combustion of fuels.

Source: Mangat, P.S., 1999. INCO Copernicus Project, IC15-CT 96-0741 Project: High Performance Materials Derived from Industrial Waste Gypsum, Co-ordinator: Prof. P.S. Mangat, Sheffield Hallam University, 1997–1999; Mangat, P.S., Khatib, J.M., Wright, L., 2006. Optimum utilisation of flue gas desulphurisation (FGD) waste in blended binder for concrete. Constr. Mater. J. Proc. ICE 1 (2), 60–68.

desulphurised wastes from a selection of countries, the type of resource, and the process used in the desulphurisation process. Table 26.4 shows the major oxide content and Table 26.5 shows the main mineralogical phases. From the information presented, it is clear that the types of materials produced vary in composition greatly and are dictated by the type of processed, raw materials used and country of origin.

Table 26.4 Major oxide content of desulphurised wastes (Mangat, 1999)

Country	Reference	Major oxide content (% mass)										
		SiO_2	Al_2O_3	Fe_2O_3	CaO_{Tot}	CaO_{free}	MgO	SO_3	K_2O	Na_2O	TiO_2	L.O.I
Poland	PL-RP	36.75	19.04	5.42	19.95	9.20	2.23	4.60	–	–	0.95	7.06
	PL-RS	41.95	21.56	6.72	13.32	6.21	2.00	2.79	–	–	0.97	6.11
	PL-P	4.04	2.32	0.94	40.07	2.72	0.86	23.66	–	–	0.20	20.32
	PL-ZP	33.11	17.22	6.88	22.48	6.15	2.96	9.65	–	–	0.80	3.15
	PL-ZZ	60.25	23.35	4.36	1.50	1.00	2.85	0.37	–	–	1.08	0.20
	PL-L	36.43	19.10	4.66	16.32	1.20	1.92	9.66	–	–	0.83	8.58
Czech Republic	CZ-ZB	61.30	25.80	3.76	1.29	0.07	1.49	–	3.10	0.50	1.14	1.74
	CZ-ZC	35.30	19.40	5.49	19.50	4.66	1.98	–	2.04	0.75	0.79	10.91
	CZ-ZF	42.40	18.00	6.06	20.80	7.45	2.29	–	2.04	0.59	0.70	1.33
	CZ-SFA	44.00	31.90	10.80	2.03	–	0.99	–	0.45	0.70	6.58	2.42
	CZ-SD	1.58	1.41	0.37	47.80	–	0.42	–	0.06	0.16	0.39	11.40
Slovak Republic	SL-F	65.86	17.42	5.96	4.03	–	1.82	1.10	2.33	1.91	–	1.80
	SL-B	48.95	10.71	3.02	30.11	–	1.48	9.80	2.33	1.16	–	4.60
	SL-FA	34.19	8.55	3.96	40.26	–	1.32	6.9	1.07	0.96	–	3.35
	SL-G	0.76	0.23	0.12	38.89	–	0.60	42.47	0.04	0.00	–	–
Holland	SDA	0.4	3.5	1.1	39.90	–	–	39.00	–	–	0.20	–
Germany	FBC-A	35.1	17.6	5.4	20.3	–	1.50	9.6	3.0	1.21		–
Denmark	FBC-B	28.9	11.3	15.7	17.1	–	1.34	3.1	1.14	0.84	–	–

Source: Mangat, P.S., 1999. INCO Copernicus Project, IC15-CT 96-0741 Project: High Performance Materials Derived from Industrial Waste Gypsum, Co-ordinator: Prof. P.S. Mangat, Sheffield Hallam University, 1997–1999; Mangat, P.S., Khatib, J.M., Wright, L., 2006. Optimum utilisation of flue gas desulphurisation (FGD) waste in blended binder for concrete. Constr. Mater. J. Proc. ICE 1 (2), 60–68.

Table 26.5 **Mineralogical composition of desulphurised wastes (Mangat, 1999)**

	Poland (PL)						Czech Republic (CZ)					Holland				Slovak Republic (SL)		
Phase	**L**	**RS**	**RP**	**ZP**	**ZZ**	**P**	**ZB**	**ZC**	**ZF**	**SFA**	**SD**	**ANH-RO**	**SDA**	**FBC-A**	**FBC-B**	**F**	**B**	**D**
Quartz	+++	+++	+++	+++	+++		++	+	+	++	+			+++	+++	++	++	+++
Mullite	+++	++	++							++	+							
Gypsum			+			+++							+					
Anhydrite		+++	+++	+++		+	+	++	+++		++	+++		+++		+++	+++	
Calcite	++	++	++	+		++	+	+	++					+			+	
Portlandite			+										+++					
Lime		++	+++	++			+	++	++					++		+++	+	
Calcium sulphite	+++										+++							
Calcium sulphate/sulphite	+																	
Iron sulphate hydroxide	++																	
Hannebachite						+++							+++					
Illite					++													
Hematite				+			+	++	++	+				+	+++			+
Halite													++					
Mica														++				
Glass							+	+	+		+			+++		+		+
Calcium-iron phosphate															++			
Feldspar							+	++	+									
Clay minerals							++	+	++									
Unburned rest							+											
Anorthite																	+	++
Opal																		+

Key: (+++)—predominant phase, (++)—medium phases, (+)—trace phases.
Source: Mangat, P.S., 1999. INCO Copernicus Project, IC15-CT 96-0741 Project: High Performance Materials Derived from Industrial Waste Gypsum, Co-ordinator: Prof. P.S. Mangat, Sheffield Hallam University, 1997–1999; Mangat, P.S., Khatib, J.M., Wright, L., 2006. Optimum utilisation of flue gas desulphurisation (FGD) waste in blended binder for concrete. Constr. Mater. J. Proc. ICE 1 (2), 60–68.

26.3.2 Evaluation of desulphurised wastes

Recent studies have identified that the varied chemical composition of wastes introduces many different reaction systems. Some of these reaction systems include calcium-sulphate reactions, pozzolanic reactions, and gypsum-fly ash–lime reactions, as well as more typical reactions occurring in plain cements (Mangat, 1999). Given the varied composition of the materials, it is important to identify test methods that best evaluate different reactions expected and allow comparisons with more common cement replacement materials, such as fly ash, slag, and silica fume. The complex nature of the materials and reaction systems also means that evaluating reactivity is difficult and using one technique may not be appropriate for all materials. Many techniques have been developed over the years that evaluate chemical, physical and mechanical properties of cement replacement materials, some of which focus on evaluating chemical properties (Mangat, 1999; ASTM, 1999) and others physical properties (BS EN 450, 1995), to determine reactivity.

26.3.2.1 Methods used to measure reactivity

Some of the main phase components of the materials produced include gypsum, anhydrite, basanite, hannebachite, glassy fly-ash particles, lime and portlandite (Mangat, 1999). The variation in mineralogical composition of the materials means that there will be many different reaction systems occurring during hydration.

The chemical methods used to measure the reactivity of desulphurised waste during a recent study included ASTM C379-565 (1999) and a method developed by KEMA (Mangat, 1999), both of which are commonly used to evaluate the pozzolanicity of a wide range of ashes. The ASTM and KEMA methods measure pozzolanicity by evaluating components of the leaching medium, which is normally a strong alkaline solution. For the ASTM method, the amount of SiO_2 and Al_2O_3 leached in the solution is recorded, and the higher the concentration, the greater the pozzolanicity of the material. Table 26.6A shows the results of the ASTM method for a selection of materials investigated.

The main difference between the ASTM and the KEMA methods is the type of alkaline solution used, which is more typical of the pore solution of actual cement, and thus more representative of what occurs during the hydration of cements. For this reason, the KEMA method should have more practical significance. The KEMA method measures the amount of Si^{4+} and Al^{3+} ions in the pore solution, and the greater the pozzolanicity of the material, the lower the concentration of leached ions in the solution. Table 26.6B shows the results of the KEMA method for a selection of materials investigated during a recent study.

Table 26.6C shows the order of reactivity for the materials shown in Table 26.6 based on the chemical methods for evaluating pozzolanicity.

The use of chemical methods can give contradictory results with regard to the degree of reactivity. This is because not all desulphurised wastes exhibit true pozzolanic properties. Some exhibit cementitious reactions that are not identifiable by the chemical methods selected. The tests also give no indication of how the materials will perform in cement-based systems.

Table 26.6 Results of ASTM and KEMA method for determining pozzolanicity (Mangat, 1999)

Component	Material					
	PL-L	PL-RS	PL-RP	PL-ZP	PL-ZZ	PL-P
(A) ASTM method						
SiO_2 (mg/L)	11.14	10.23	4.40	8.50	12.84	0.80
Al_2O_3 (mg/L)	4.35	4.30	2.85	5.44	4.62	0.68
(B) KEMA method						
Si^{4+} (mg/dm^3)	1.255	1.585	1.895	1.76	3.780	4.915
Al^{3+} (mg/dm^3)	1.045	0.940	1.050	1.035	1.340	0.385
(C) Degree of pozzolanicity						
	Most reactive				Least reactive	
ASTM	PL-ZZ	PL-L	PL-RS	PL-ZP	PL-RP	PL-P
KEMA	PL-L	PL-RS	PL-ZP	PL-RP	PL-ZZ	PL-P

Source: Mangat, P.S., 1999. INCO Copernicus Project, IC15-CT 96-0741 Project: High Performance Materials Derived from Industrial Waste Gypsum, Co-ordinator: Prof. P.S. Mangat, Sheffield Hallam University, 1997–1999.

The activity index test defined in BS EN 450 (1995), which is commonly used to determine the pozzolanicity of fly ash, has been used to measure the reactivity of desulphurised wastes (Mangat, 1999).

Table 26.7 shows the results of the activity index tests carried out on a selection of desulphurised wastes (Mangat, 1999). Materials such as PL-RP and PL-ZP generally satisfy the requirements of BS EN 450 (1995). Materials that did not meet the requirements of BS EN 450 include PL-P, SL-G, and G, which is expected as they are predominantly calcium sulphate/sulphite based.

The measurement of strength appears to provide a more practical method to distinguish the reactive properties of different desulphurised waste materials and allows comparisons to be made with other more common cement replacement materials such as fly ash, ground granulated blast furnace slag, and silica fume.

26.3.3 Classification of desulphurised wastes

The evaluation of desulphurised wastes, based on activity index testing, during recent studies has led to classification of desulphurised wastes as follows (Mangat, 1999):

1. Nonsiliceous and deleteriously reactive: These materials consist of FGD gypsum from wet FGD processes, which when used in cement-based materials has a detrimental effect on binder development, such as strength loss. The composition of these materials is normally calcium sulphate in the form of gypsum. The SO_3 content of these materials is approximately 35–50%.
2. Nonsiliceous and nonreactive: These wastes are from dry and semidry flue-gas cleaning. These materials in general consist of calcium sulphate/sulphite in nonreactive form. The SO_3 content of these materials is approximately 20–30%.
3. Siliceous and nonpozzolanic active: These wastes were usually bottom ashes from FBC processes. The nonreactive nature of these materials stems from the relatively coarse nature of the particles. The SO_3 content of these materials is approximately 10–15%.
4. Siliceous and pozzolanic active: In general, these materials are fly ashes from FBC and from dry and semidry flue-gas cleaning. In the development of binders, these materials had positive effects on properties such as strength. The SO_3 content of these materials was approximately 10% or less.

Table 26.7 **Results of activity index method for determining reactivity**

	Activity index (relative strength, %)[a]							
Material	**UK limits (%)**	**PL-P**	**PL-ZP**	**PL-RP**	**SL-G**	**SL-F**	**FA[b]**	**G[b]**
28 day	75	46.26	82.86	82.67	24.61	76.87	76.87	38.87
90 day	85	50.71	82.11	78.57	23.45	81.90	86.71	31.04

[a] Strength relative to a reference mix of 100% cement at the desired age.

[b] FA designates UK fly ash and G designates UK gypsum.

These broad classifications are based on the known composition of each material and results of pozzolanicity tests carried out (Mangat, 1999). The first category had negative effect on relative strength, that is, a reduction in strength gain, and showed signs of significant sample deterioration. Category 4 had beneficial influences on strength development, with some waste exhibiting a relative strength of 110% in relation to a reference mix of 100% CEM1.

26.3.4 Simulated desulphurised waste (FA-G blends, SDW)

Current investigations (Mangat, 1999; Mangat et al., 2006) have shown that cement-based materials containing high levels of desulphurised wastes exhibit physical, mechanical, and durability properties equal or superior to reference cements, and cements containing more common replacement materials such as fly ash and slag. The variation in materials produced from different processes makes it difficult, and impracticable, to assess and compare properties of actual desulphurised wastes in construction materials.

Attempts have been made to simulated desulphurised wastes (SDWs), by blending fly ash and gypsum (FA-G blends), to assess the influence of chemical composition and predict the performance of desulphurised wastes in construction materials. The gypsum contents ranged from 0% to 100% to achieve SO_3 contents ranging from 1.75% to 49.2% by weight. It was shown that the relationship between SO_3 content and compressive strength of pastes containing SDW could predict the short- and long-term theoretical strength of cement-based materials containing actual desulphurised wastes with reasonable accuracy (Mangat et al., 2006).

Fig. 26.1 shows the relationship between 90-day compressive strength and the chemical composition of actual and simulated desulphurised wastes, in relation to (A) SO_3 content and (B) $SiO_2 + Al_2O_3$ (S + A) content. It appears that calcium sulphate

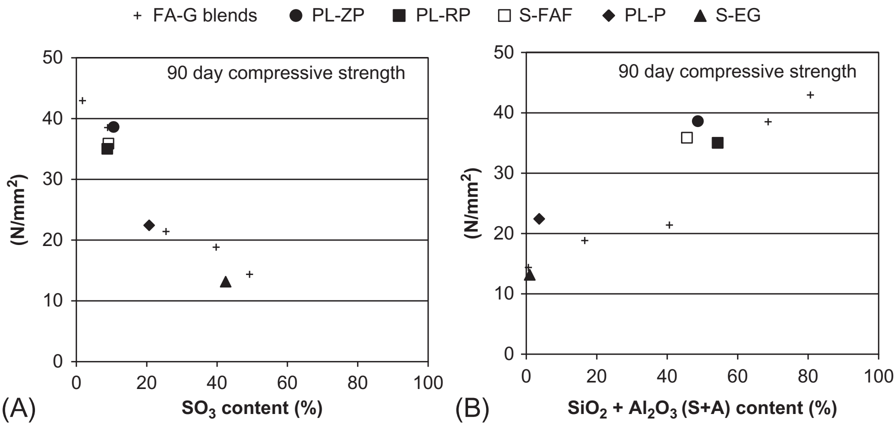

Fig. 26.1 Strength of mortars containing actual and simulated desulphurised wastes in relation to (A) SO_3 content and (B) S+A content. (Mangat et al., 2006).

content, associated with the gypsum, influences strength to a greater extent than S + A content, associated with fly ash. The relationship between strength and SO_3 content appears to provide a more accurate method of predicting the strength of actual desulphurised wastes.

To investigating the influence of desulphurised-waste content in cement-based materials, it is impractical to evaluate all simulated desulphurised wastes (FA-G blends) available. Studies have focused on a typical SDW to investigate the influence of cement replacement level (Mangat et al., 2006). This typical SDW was based on typical SiO_2-Al_2O_3-$CaSO_4$-based desulphurised wastes, rather than calcium sulphate-based desulphurised waste.

Table 26.8 shows the SO_3 content of some typical SiO_2-Al_2O_3-$CaSO_4$-based desulphurised waste evaluated. Table 26.9 shows the compressive strength of three actual desulphurised wastes and two SDW with SO_3 contents ranging between 8% and 11.3%.

Based on these investigations, a typical SDW, an optimum FA-G blend of 85% fly ash and 15% gypsum with SO_3 content 8.87% by weight of replacement material was selected. This was used to evaluate the influence of desulphurised-waste content on the properties of cement-based material, such as strength, chemical shrinkage, porosity and pore-size distribution, and sulphate resistance. These mixes are referred to as C-SDW blends.

Table 26.8 **Typical SO_3 contents of SiO_2-Al_2O_3-$CaSO_4$-based desulphurised wastes**

Desulphurised waste	PL-RP	PL-RS	PL-ZP	PL-L	CZ-ZC	CZ-ZF	FBC-A	Average
SO_3 content (%)	8.78	2.79	10.57	9.66	6.52	6.52	9.6	8.61

Table 26.9 **Compressive strength of actual and typical SDW with similar SO_3 contents**

Replacement material	SO_3 content (%)	Compressive strength (N/mm^2)		
		7 days	28 days	90 days
$100_{PL\text{-}RP}$ (actual)	8.8	14.3	23.2	35
$85_{FA}15_G$ (typical)	8.9	12.7	23.3	38.5
$100_{SL\text{-}F}$ (actual)	9.1	12.8	27	35.9
$100_{PL\text{-}ZP}$ (actual)	10.6	12.5	26.7	38.6
$80_{FA}20_G$ (typical)	11.3	10.4	18.0	35.3

All mixes are based on 75% CEMI and 25% replacement material.

26.4 Influence of FGD on concrete properties

26.4.1 General

As an integral part in the development of cement, without gypsum the reaction of C_3A is extremely rapid and leads to sudden stiffening of the hydrating paste (Neville, 1995a). The gypsum is added on the basis that it reduces the amount of C_3A and C_4AF hydrated in the preinduction period thus regulating the hydration of cement (Collepardi et al., 1978). After the initial stages, the gypsum reacts with the C_3A to form ettringite (AFt phase) until all the gypsum is consumed. Any remaining C_3A present may then react with the ettringite to form monosulphate (AFm phase). Ideally, a balance of gypsum and C_3A should be attained so the consumption of both is within the first 5–10 h. An imbalance in these can lead to a delay or an excessive formation of ettringite once the hydrating paste is no longer in a plastic state causing damage to the cement matrix due to internal expansions.

Lerch (1946) reported that there are three major factors influencing gypsum content in cement: C_3A content, alkali content (Na_2O, K_2O), and fineness of ground clinker. The optimum SO_3 content is based on a combination of sufficient retardation, highest strength, lowest shrinkage, and minimum expansion.

Since there are many variables, the selection of optimum sulphate content is not straightforward. If higher strengths are required, the clinker can be ground finer, which would increase its surface area and allow more C_3A to be available for reaction. To regulate setting, the gypsum content would need to be increased to facilitate the increase in C_3A content available for reaction (Lerch, 1946). Methods for calculating the optimum SO_3 content in cement have been presented, which are based on C_3A, alkali content, and fineness of the clinker (Haskell, 1959; Ost, 1974). Eq. (26.1) shows one such relationship for determining SO_3 content (Ost, 1974).

$$\%SO_3 = 0.556(\%Na_2O) + 0.00177(\text{fineness}) - 0.107(\%Fe_2O_3) - 3.6004 \quad (26.1)$$

Gypsum in cement not only retards the reaction of clinker minerals in cement hydration, but also accelerates the reaction of minerals such as tricalcium silicate (alite-C_3S). Bentur (1976) reported results on alite pastes containing gypsum contents ranging from 0% to 9%. The C_3S and CH contents were evaluated to determine the degree of hydration. The results showed that at all ages, the degree of hydration for pastes containing 2%, 4%, and 8% gypsum was similar but was generally higher than that of pure C_3S pastes. The difference in the degree of hydration between the pure C_3S pastes and the pastes containing gypsum reduced with age. At 90 days, all pastes had undergone approximately 85% hydration. With regards to strength, when the gypsum content exceeded 2%, the strength of the paste was significantly reduced, especially during the first 2 days of hydration. This was attributed to the change in the quantity and quality of the calcium silicate hydrates (C-S-H) formed during hydration.

Increasing the gypsum content increased the amount of C-S-H formed but decreased its intrinsic strength. Increasing the gypsum content resulted in the formation of C-S-H gel that had a higher C/S ratio, which was linearly related to the intrinsic strength of the C-S-H gel.

Some calcium sulphate-based desulphurised contain large quantities of the sulphite mineral phase hannebachite. The reaction of hannebachite into gypsum is reported to be very slow and assumed to be negligible (Mangat, 1999; Khatib et al., 2007b, p. 395), and also reduces the sulphate content of the mix. The presence of sulphite and the lower sulphate content of these materials could explain why long-term strength gain is better than in pure calcium sulphate-based materials.

The variability in wastes from dry and semidry processes means it is difficult to identify typical reactions that occur, unlike in similar coal combustion wastes such as fly ash. However, because wastes from such processes contain fly ash, it is possible that they possess some pozzolanic traits, in addition to cementitious properties. Investigations carried out on such wastes are limited, however, it has been shown that the predominant phases formed in cement-based materials containing desulphurised waste are calcium silicate hydrates (C-S-H), ettringite (AFt), and monosulphate (AFm) as with other pozzolanic materials such as fly ash (Demirabas and Aslan, 1999; Mangat, 1999). Investigation was carried out on the reactivity of coal combustion wastes, and similar conclusions were derived, in that their compositions varied greatly, which in turn influences their reactivity and their utilisation (Iribarne et al., 2001; Bernado et al., 2000). Combustion ashes are used not only in cement systems, but also in noncement systems where blends of different wastes have been investigated to produce binders with strengths similar to that of normal cements (Mangat et al., 2006; Khatib et al., 2007b, p. 395). The main phases formed included C-S-H and ettringite. Wastes with high SiO_2 and Al_2O_3 contents increase the pozzolanic reactions and result in the formation of additional C-S-H and C-A-H.

Although some FGD wastes possess similar chemical compositions, with respect to major oxide content, the way in which the major oxides are bound within the material varies. The presence of CaO in the waste can exist bound with other compounds or present as free lime, the latter being more important when evaluating volume stability. In cement, the presence of free lime in the clinker, which is intercrystallised with other compounds, is partially exposed to water prior to setting. The reaction of free lime can lead to an increase in volume change. This can affect the structure of the hardened cement paste if expansions associated with the reaction of free lime occur after setting has taken place (Neville, 1995a). Larger amounts of CaO and smaller amounts of free lime can also make more lime available for cementitious reaction, and thus increase strength.

The majority of dry and semidry FGD wastes exhibit other pozzolanic traits, such as retarded early strength and increased long-term strength compared to plain cements (Smith, 2006; Ghafoori and Garcia Mora, 1998). Other properties affected include an increase in water demand (Schiller and Eklerbrock, 1992), which was attributed to the fineness of the material. Some bed ashes are extremely coarse and may be more suitable for aggregate replacement in mortars and concrete rather than cement replacement materials.

26.4.2 Chemical shrinkage

Chemical shrinkage is evaluated based on two measurable quantities: external chemical shrinkage (ECS) and total chemical shrinkage (TCS) as reported by Justness (Justnes et al., 1995; Wright et al., 2007). ECS is fundamentally a measure of the change in external volume due to the chemical reactions taking place during the early stages of hydration. TCS is a measure of the pastes' ability to react and thus harden in the presence of moisture, which is generally called hydration. TCS includes the external volume changes (ECS) and internal volume changes.

The chemical shrinkage of a cement paste evaluated by Reyniers and van Loo showed that ECS and TCS are equal up to 8 h (Reyniers and Van Loo, 1993). After this, ECS appears to stop once the paste structure is rigid enough to resist the internal contracting forces. The TCS continues to increase beyond 8 h as hydration products are formed.

There is limited knowledge on the influence of mineral additives on chemical shrinkage using current techniques (Khatib et al., 2011; Wright et al., 2007). It has been reported that with some calcium sulphate-based materials, there is a reduction in the rate of chemical shrinkage during the first 12 h. It has been observed that the introduction of calcium sulphate-based materials retards the formation of early hydration products, which results in a reduction in chemical shrinkage (Khatib et al., 2011).

ECS was much higher than that observed in reference cements, which was attributed to the retardation of the hydration process, which increased setting times and reduced the rate of hardening. Therefore, longer periods are needed to achieve sufficient strength to resist the internal contracting forces, and thus more shrinkage could take place than in the reference mix. The ECS continues to increase until the strength of the paste is sufficient to resist the internal contracting forces. There is an argument to suggest that if hydration is retarded the products will not be formed that cause the shrinkage, hence resulting in a lower ECS.

The influence of calcium sulphate-based materials on TCS has been reported to show a large reduction in TCS, which was attributed to the retarding effects of the gypsum (Wright et al., 2007). Fig. 26.2 shows a large reduction in TCS due to the replacement of cement with calcium sulphates G and SL-G, which was attributed to the retarding effects of the gypsum.

It has been reported that mixes containing SiO_2-Al_2O_3-$CaSO_4$-based desulphurised wastes exhibit an increase in ECS, which suggests an increase in setting times, or that the materials accelerate the early hydration reactions, which increases the amount of products formed (Khatib et al., 2011). The majority of materials appears to be contributing to the reaction process either directly or by increasing the reaction of cement by increasing the surface area of cement particles available for reaction. The ECS appears to level off between 12 and 24 h. In similar pozzolanic materials, such as fly ash, longer setting times and low strengths compared to reference cements have been reported (Costa and Massazza, 1983). The influence of fly ash on cement has

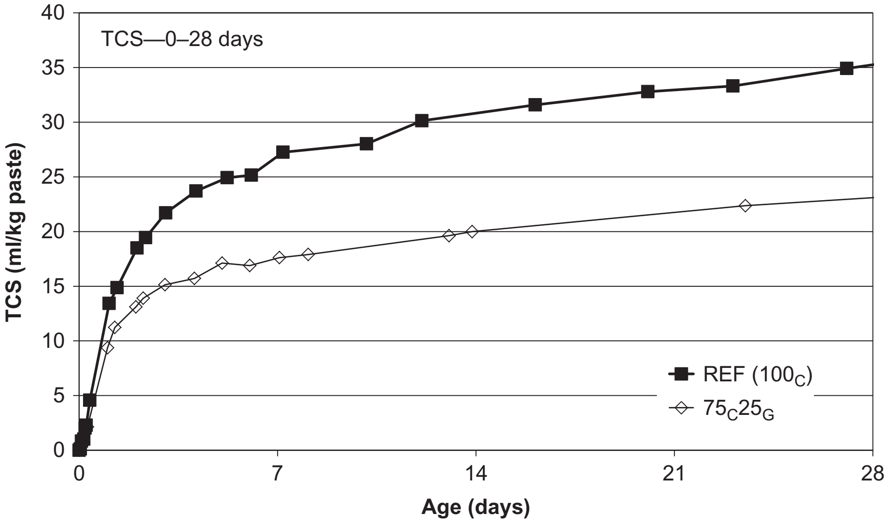

Fig. 26.2 TCS of paste containing calcium sulphate desulphurised waste (Wright et al. 2007).

been shown to retard early C_3S and C_3A hydration, which is responsible for setting and hardening (Fajun et al., 1985). In addition, the materials contain small amounts of calcium sulphate and have SO_3 contents of approximately 10%, which could have a retarding effect.

The long-term chemical shrinkage (TCS) for all pastes was reported to be similar to the reference mix as shown in Fig. 26.3 (Wright et al., 2007). There was a slight increase in the rate of change in TCS with time. This was attributed to the pozzolanic

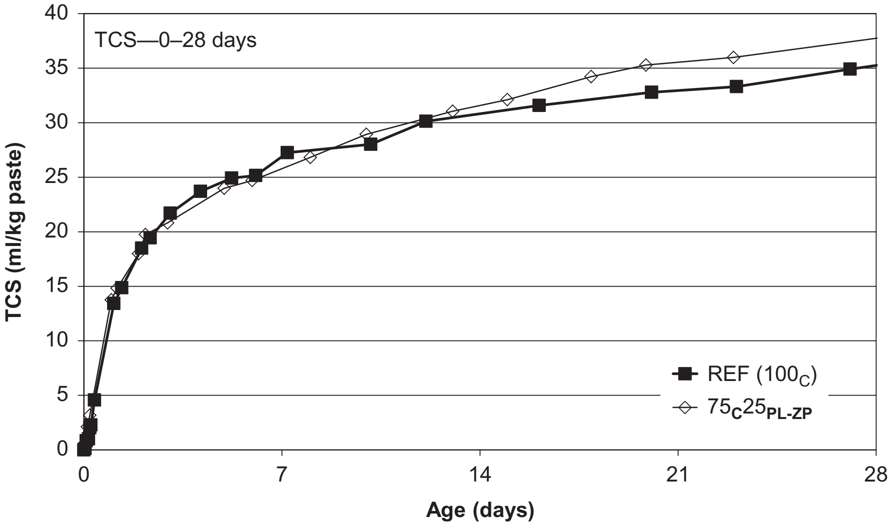

Fig. 26.3 TCS of pastes containing SiO_2-Al_2O_3-$CaSO_4$-based desulphurised wastes (Wright et al. 2007).

properties of the materials, which contribute to the hydration process by changing the amount or type of hydration products formed, which would promote chemical shrinkage. A change in the hydration products formed could also result in a change in the density of the hardened material. For instance, by introducing a pozzolanic material, such as fly ash, the amount of CH formed reduces, and additional C-S-H or C-A-H are formed, which increase the density of the paste (Taylor, 1997). The majority of SiO_2-Al_2O_3-$CaSO_4$-based desulphurised wastes is not truly pozzolanic and possesses relatively diverse chemical characteristics, therefore, it is difficult to identify what products are formed. The materials also contain CaO contents around 20%, so it is likely that cementitious reactions were taking place in addition to pozzolanic reactions.

The evaluation of simulated desulphurised wastes, which consist of different fly ash–gypsum (FA-G) blends, has shown that TCS during the first 12 h was reported to be similar for all FA-G blends (Khatib et al., 2010).

During the first 7 days of hydration, increasing the gypsum content in the FA-G blend generally decreased TCS. After 7 days, increasing the gypsum content appeared to increase TCS for gypsum contents of less than 25% (SO_3 13.62%). It has been reported that the reactivity of fly ash could be improved in the presence of calcium sulphate, which has been shown to improve physical and mechanical properties of cement-based materials (Uchikawa, 1986; Wild et al., 1990; Poon et al., 2001; Shiyuan, 1982).

26.4.3 Porosity and pore-size distribution

The measure of porosity and pore structure, with respect to pore diameter, extent, and shape, is extremely important when determining the performance and utilisation of materials. The measure of porosity and pore-size distribution is well documented for the performance of mineral additives in cements and their effects on physical, mechanical, and durability properties. Although relatively novel, recent studies have reported on the influence of desulphurised wastes on porosity and pore structure of cement pastes (Khatib et al., 2013a, p. 622; Khatib et al., 2013b, p. 201).

The effects of gypsum on the progress of hydration affect both early and long-term strength, which is linked to the pore structure within the hydrated material (Neville, 1995a). Gypsum is added to regulate setting by suppressing the hydration of C_3A. If the gypsum content in the mix is in excess of the amount that reacts with C_3A during setting, a slow expansion can occur (Neville, 1995b). The expansive effects can cause possible damage to the hardened material, which can alter total porosity, and pore structure. It has been reported that increasing SO_3 content in cement pastes between 0.4% and 5% by adding gypsum content increased porosity, which also coincided with a decrease in strength (Sorka and Abayneh, 1986). Correlating porosity, degree of hydration, SO_3 content, and strength showed that porosity was closely related to the degree of hydration, and independent of SO_3 content, and the linear relationship between strength and degree of hydration was dependent on SO_3 content. The increase in SO_3 content had a positive effect on strength development for mixes with similar porosities and degree of hydration. It was reported that the increase in strength with increasing SO_3 content for mixes with similar porosity occurred because of an increase in pore refinement rather than reduction in total porosity.

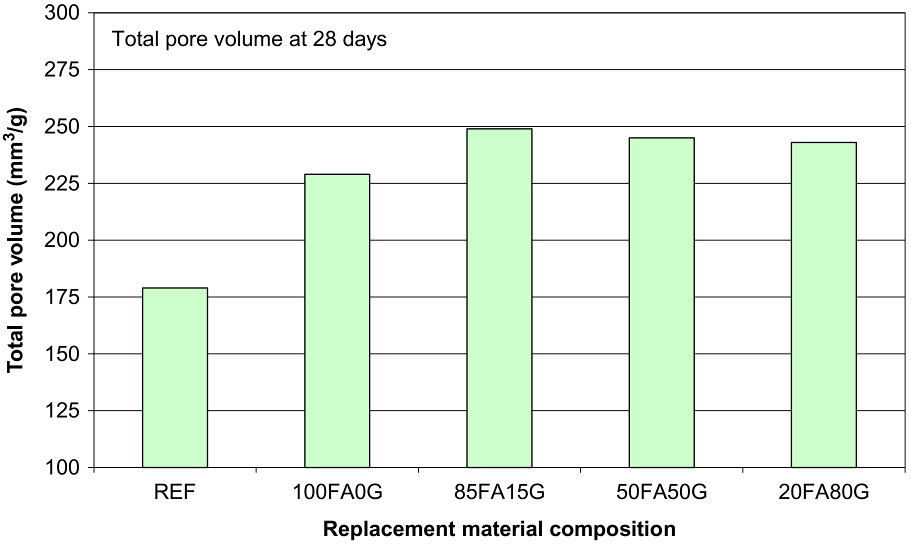

Fig. 26.4 Influence of FA-G composition on total pore volume.

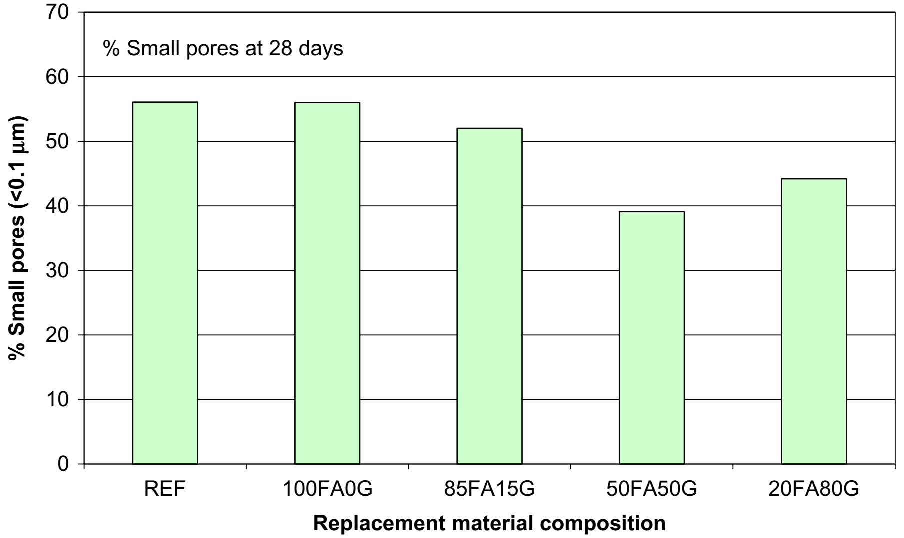

Fig. 26.5 Influence of FA-G composition on the percentage of small pores (>0.1 μm).

Studies to assess the influence of FA-G blends on porosity and pore structure of cement-based materials have shown that there is an increase in the total pore volume (TPV) of the paste as shown in Fig. 26.4 (Khatib et al., 2013a, p. 622; Khatib et al., 2013b, p. 201).

An increase in TPV during the first 28 days was accompanied by a decrease in pore refinement, which constituted an increase in the threshold diameter (TD) as shown in Fig. 26.5 (Khatib et al., 2013a, p. 622; Khatib et al., 2013b, p. 201).

Manmohan and Mehta (1981) reported that replacing cement with fly ash increased porosity and decreased pore refinement at 28 days, and decreased porosity and increased pore refinement at 365 days, compared to a reference mix of 100% cement. For FA-G blends evaluated, a blend of 85% fly ash and 15% gypsum appeared to be the optimum blend in terms of minimising TPV and increasing pore refinement. Uchikawa (1986) reported that the reactivity of fly ash could be improved with the addition of sulphate, which introduces SO_4^{2-} and reacts with the Al_2O_3 in the fly ash. This breaks down the structure of the glass and crystalline phases. The increase in reactivity would result in more reaction products formed, which would fill the open pores and thus decrease the TPV of the paste (Khatib et al., 2013a, p. 622; Khatib et al., 2013b, p. 201).

Wild et al. (1990, 1995) reported that the porosity and pore-size distribution of fly ash–lime mixes could be improved by the addition of gypsum. However, excessive gypsum can result in a retardation of the hydration process as the ettringite formed on the fly-ash particles increases, which temporarily retards the reaction with lime (Wild et al., 1990).

One indication of the alteration of the pore structure was the appearance of a secondary rounded peak on the pore-size distributions (PSDs) (Khatib et al., 2007a,b,c; Khatib et al. 2013b).

26.4.4 Compressive strength

Compressive strength is considered one of the most important properties of concrete, and is directly related to the degree of hydration and structure of the hardened material, which ultimately influences physical, mechanical, and durability properties of the concrete. Concrete is usually specified by strength as it gives an indication of the quality of the concrete. Mineral additives, such as fly ash, can retard or accelerate the hydration reactions and change the rate, type, and amount of hydration products formed resulting in changes in the microstructure of the hardened material that can affect strength (Khatib et al. 2016).

Lerch (1946) reported that gypsum in cement generally increased strength, with optimum gypsum contents based on pastes that exhibit a combination of sufficient retardation, highest strength, lowest shrinkage, and minimum expansion. It has also been reported that gypsum, or more correctly sulphate, can be added to blended cements that contain fly ash to improve their performance (Poon et al., 2001; Wild et al., 1990).

One of the main mineral components in cement hydration is tricalcium silicate (C_3S). Bentur (1976) reported that increasing gypsum content resulted in the acceleration of C_3S reactions. At 3 and 7 days, a maximum compressive strength was achieved for mixes containing 2% gypsum. Increasing the gypsum content above 2% resulted in a decrease in strength. Increasing the gypsum content increases the C/S ratio of the C-S-H formed, which corresponds to a decrease in the intrinsic strength of the C-S-H gel. This occurs due to excessive gypsum, which reacts with the C-S-H gel formed and changes its structure.

Shiyuan (1982) reported that the strength of cement pastes containing 20% fly ash could be improved by increasing the gypsum content. Early strength was reduced in all mixes, which was attributed to a reduction in the C_3A hydrated and the formation of poor quality C-S-H gel. However, with age there was a constant strength gain, and mixes containing 4% gypsum exhibited a large strength increase between 8 and 90 days. Between 8 and 90 days, there was an increase in C_3S and CH reacted, indicating that the gypsum accelerates the hydration of C_3S and pozzolanic reactions, and XRD and SEM work showed that in mixes containing 4% gypsum the bond between the fly ash grains and the hydrates around them were improved, which contributed to the increase in strength. When the gypsum content exceeded 4%, it was observed that the strength was significantly reduced at all ages due to the reduction in C_3S and C_3A hydrated.

Poon et al. (2001) reported that anhydrite (Ca_2SO_4) was more beneficial to early strength development than gypsum ($Ca_2SO_4{\cdot}H_2O$) when the same amount of addition was used, which was attributed to anhydrite having a greater amount of SO_3 that is available for reaction. However, when gypsum and anhydrite are proportioned to give an equivalent amount of SO_3, gypsum is more beneficial for increasing long-term strength, which was attributed to the rate at which sulphate ions are supplied for reaction, as gypsum supplies sulphate ions for reaction more quickly than anhydrite.

The large variation in composition of coal combustion wastes from dry and semidry desulphurisation process means that the strength properties of binders containing such wastes are significantly different, as discussed earlier. Attempts to investigate the strength of such materials in cement-based systems have been reported but are process specific (Cornelissen, 1991; Drottner and Havlica, 1997; Bloem and Sciarone, 1991, Khatib et al. 2016). An example of strength development in Fig. 26.6.

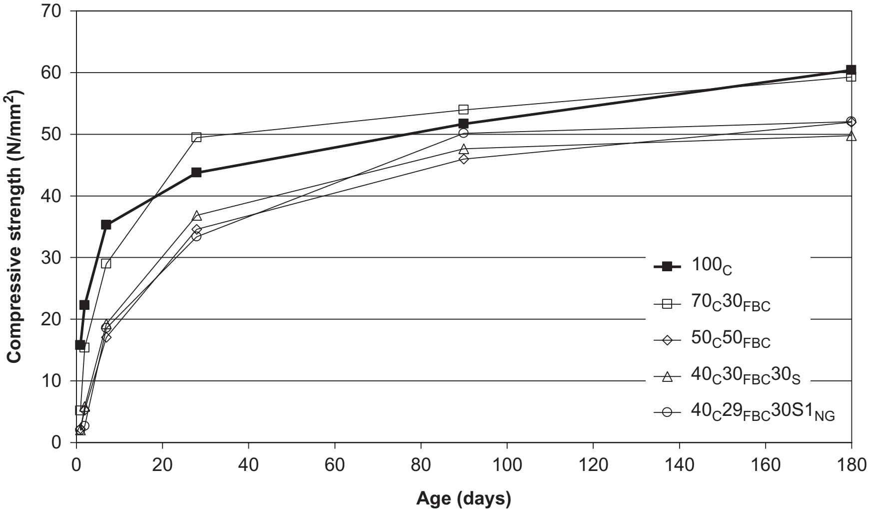

Fig. 26.6 Compressive strength of C-FBC-S-NG mixes (Drottner and Havlica, 1997).

26.4.5 Sulphate resistance

The presence of sulphates such as sodium (Na_2SO_4) and magnesium ($MgSO_4$) in groundwater and seawater can reduce the service life of concrete structures. Generally, sulphate attack is associated with the formation of gypsum and ettringite, which is reported to result in large expansions (Wee et al., 2000; Hime and Mather, 1999). Na_2SO_4 also reacts with the calcium aluminate hydrate (C-A-H) phases to form ettringite, as shown by the reaction in Eq. (26.2).

$$\begin{aligned} &2\left(3CaO\cdot Al_2O_3\cdot 12H_2O\right)+3\left(Na_2SO_4\cdot 10H_2O\right)\\ &\rightarrow 3CaO\cdot Al_2O_3\cdot 3CaSO_4\cdot 31H_2O+2Al\left(OH\right)_3+6NaOH+17H_2O \end{aligned} \tag{26.2}$$

The sodium sulphate (Na_2SO_4) reacts with the CH formed during hydration, and deposits gypsum as shown in Eq. (26.3).

$$Ca\left(OH\right)_2+Na_2SO_4\cdot 10H_2O\rightarrow CaSO_4\cdot 2H_2O+2NaOH+8H_2O \tag{26.3}$$

Magnesium sulphate ($MgSO_4$) reacts not only with the CH and C-A-H phases but also with the calcium silicate hydrate (C-S-H) phases, as shown in Eq. (26.4).

$$3CaO\cdot 2SiO_2\cdot aq+MgSO_4\cdot 7H_2O\rightarrow CaSO_4\cdot 2H_2O+Mg\left(OH\right)_2+SiO_2\cdot aq \tag{26.4}$$

The sulphate resistance of concrete can sometimes be improved by replacing cement with pozzolanic materials, such as fly ash (Khatib, 1991; Mangat and El-Khatib, 1992). The pozzolanic material dilutes the C_3A present in the cement and reduces the amount of CH produced. Materials such as fly ash can also improve the transport properties of the hydrated material by reducing permeability, which also increases the sulphate resistance of cement-based materials.

The influence of SDW content on the sulphate resistance of mortar has been reported based on a typical SDW consisting of an optimum FA-G blend of 85% fly ash and 15% gypsum (Khatib et al., 2008b). It was reported that the replacement of cement with SDW has a significant effect on the sulphate resistance of the mortars cured in sodium sulphate.

The visual inspection of C-SDW mortar samples immersed in sodium sulphate has been reported. Fig. 26.7 shows that reference cements undergo significant deterioration, especially around the ends, edges, and corners, whereas, C-SDW mortars show very little sign of deterioration after 180 days of immersion (Khatib et al., 2007c).

Deterioration of the reference samples took the form of faint cracking after 90 days followed by significant cracking, loss of material around corners and edges, and deformations between 90 and 180 days, and total destruction after 210 days. A similar pattern was observed for the reference mix although significantly delayed until total destruction after 390 days. After 450 days, in the sample containing 20% SDW, signs of hairline cracking were observed on the side faces. Samples containing SDW contents above 20% showed no visual sign of deterioration.

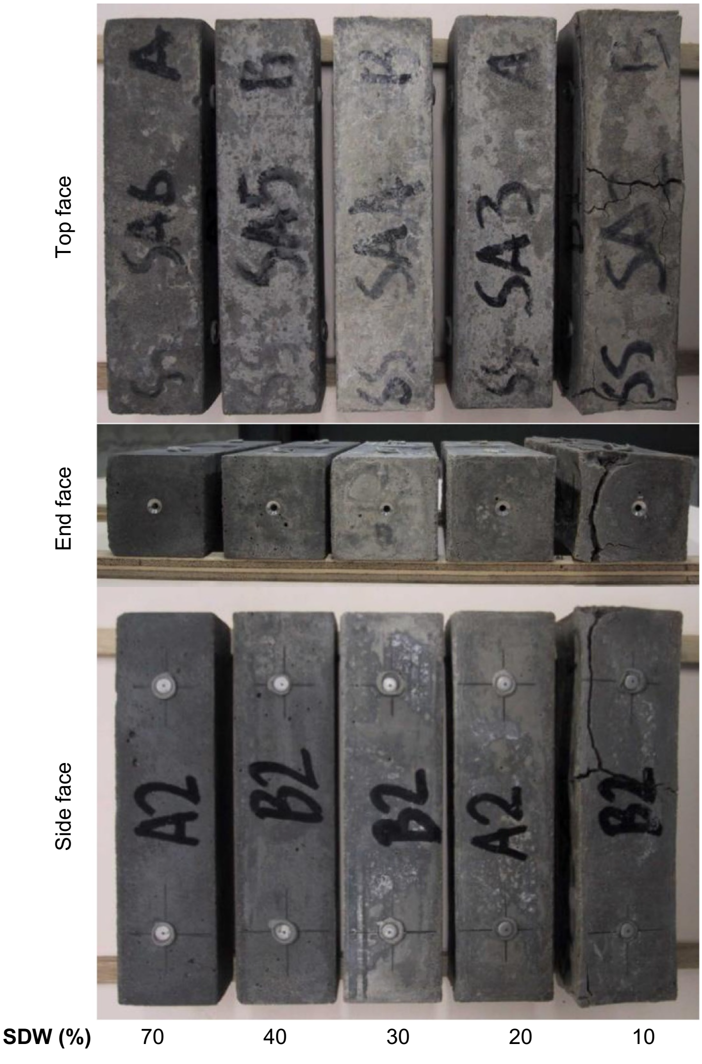

Fig. 26.7 Appearance of C-SDW mortars immersed in sodium sulphate (2.2% Na_2SO_4) for 365 days.

The visual inspection of C-SDW mortar samples immersed in magnesium sulphate has been reported. Fig. 26.8 shows the appearance of C-SDW mortars immersed for 450 days in magnesium sulphate (Khatib et al., 2008a, p. 21).

During the first 60 days of immersion, a white layer formed on the surface of the reference samples, and became thicker with time. No cracking was observed with the reference mix after 150 days, however, the presence of the white layer may have masked the start of cracking. After 180 days, the edges and corners of the reference mix had started to soften, which resulted in loss of material on handling. Slight cracking was observed and was more prominent around the edges and corners. This

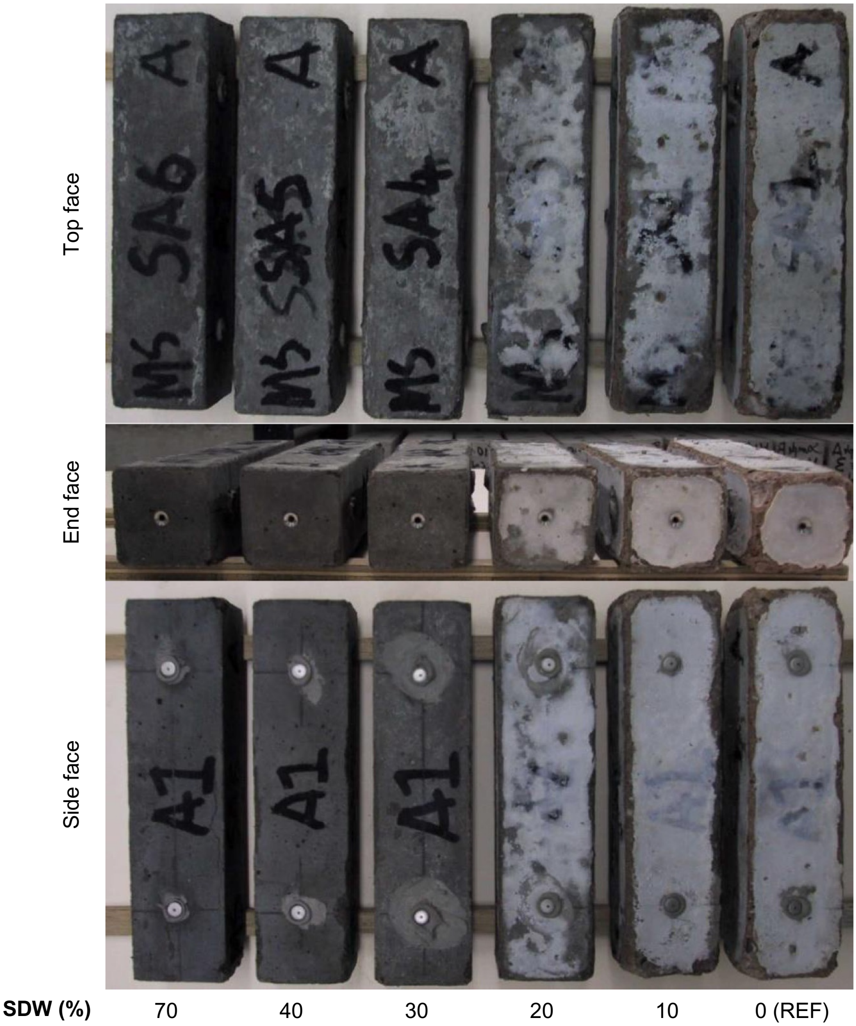

Fig. 26.8 Appearance of C-SDW mortars immersed in magnesium sulphate (2.44% $MgSO_4$) for 450 days.

deterioration process was also observed for the mortar containing 10% SDW, but was delayed by approximately 30 days. This coincides with the delayed increase in weight and length change. Increasing the SDW content above 10% appeared to reduce the deterioration process. After 450 days, deterioration was almost nonexistent for mixes containing 30% SDW and above.

Irrespective of type of sulphate medium, C-SDW blends appeared to improve sulphate resistance, which could be attributed to the chemical and mineralogical composition of the SDW. The replacement of cement with SDW reduces the C_3A and CaO content of the mix available for reaction, and the fly ash in the SDW undergoes long-term pozzolanic reactions, which consumes CH, one of the main reaction products in sulphate attack, to form additional cementing C-S-H and low calcium C-A-H phases (Lea, 1998).

The failure mechanism due to exposure to sulphate is very complicated and is dependent on many factors. The formation of ettringite during sulphate attack is generally associated with expansive failure, whereas gypsum formation tends to lead to a decalcification of the C-S-H resulting in a loss of strength. Attempts have been made to predict the sulphate resistance of concrete containing replacement materials such as fly ash, as shown below in Eq. (26.5), which assesses the calcium and iron oxide content of the fly ash (Dunstan, 1980).

$$R = \frac{\mathrm{CaO} - 5}{\mathrm{Fe_2O_3}} \tag{26.5}$$

It was reported that increasing the CaO content in the fly ash above 5% decreases sulphate resistance and increasing the Fe_2O_3 content improves sulphate resistance by reducing the expansive nature of the alumina-sulphate reactions. Values of '*R*' less than 1.5, for 25% cement replacement, were shown to increase sulphate resistance, whereas '*R*' values greater than 3 decreased the sulphate resistance when exposed to sodium sulphate. If the samples were exposed to aggressive solutions, such as magnesium sulphate, it was recommended that the limits for '*R*' should be reduced by 0.5, that is, an '*R*' value less than 1.0 would suggest an increase in sulphate resistance.

The '*R*' factor for the SDW evaluated was 1.54, which indicates the SDW would have no effect on improving sulphate resistance based on the limits above. However, the investigation clearly shows that when cement was replaced with SDW, the sulphate resistance was significantly improved, especially when the SDW content was increased above 10%. The SDW is predominantly fly ash, with an '*R*' value of 0.38, which complements the improvement in sulphate resistance associated with the C-SDW mortars investigated. It has been reported that the measure of chemical composition alone, with the '*R*' factor is insufficient to determine the sulphate resistance of the fly ash cements, but also dependent on the actual fly ash content (Khatib, 1991). Generally, the optimum fly ash content lies between 20% and 30%.

Tikalsky and Carrasquillo (1992) reported that fly ash with high amounts of calcium oxide and amorphous calcium aluminate were more susceptible to sulphate attack than concretes containing low lime fly ash (ASTM class F). Although the correlation between sulphate resistance and chemical and/or mineralogical composition were not conclusive, the major factors behind improving sulphate resistance appear to lie in reducing the CaO content, and increasing the presence of crystalline phases, such as mullite, quartz, ferrite spinal, and hematite.

Manmohan and Mehta (1981) reported that the sulphate resistance of cement was also dependent on the type of aluminate phases formed in the hydrated material before the samples were immersed in sulphate solutions. The formation of expansive aluminate phases such as ettringite occurred while the mix was in a plastic state, and thus the hydrating material accommodated the volume changes associated with these expansions. Furthermore, ettringite is stable in sulphate environments, which will also improve sulphate resistance. If monosulphate and C-A-H phases are present before immersion, the exposure to sulphate environments results in the formation of ettringite when the material is in a hardened state, and the resulting expansions can result in deterioration through cracking and spalling (Collepardi, 2001).

The SDW evaluated contains 15% gypsum, which has been reported to improve performance of fly ash cement. Freeman and Carrasquillo (1995) reported that the sulphate resistance of cement containing high lime fly ash (ASTM class C) could be improved with the addition of gypsum. Replacing the total cementitious content (cement + fly ash) with 2%, 6.5%, and 13.2% gypsum resulted in an improvement in the sulphate resistance properties compared to the mixes containing just fly ash. The sulphate resistance was presented with respect to the SO_3/C_3A of the mix, with an optimum blend achieved when the SO_3/C_3A was between 0.35 and 0.6. Once the SO_3/C_3A exceeded 0.6, the sulphate resistance generally reduced, and in excessive amounts samples were at risk from internal sulphate attack.

Mehta (1986) reported that if ettringite was formed prior to immersion in the sulphate environment, the expansion associated with sulphate attack was reduced. Therefore, in systems where gypsum has been added, the increased SO_3 content may accelerate the formation of expansive products such as ettringite during the early stages and improve sulphate resistance.

In addition to improving sulphate resistance through changing the reaction properties of cement-based materials, fly ash can also improve sulphate resistance through changes in the microstructure of the hydrated material (Poon et al., 2000; Bilodeau and Malhotra, 2000). Hughes (1985) reported that the physical aspects of sulphate attack were governed by a combination of the ability of the sulphate ion to diffuse through the pore structure, as reflected by the permeability measurements, and the ability of the sulphate ion subsequently to gain access to the susceptible minerals, determined by the entry size of the pores associated with them. Increasing the SDW content has been shown to increase porosity of pastes and increase pore refinement. The percentage of small pores lying below 0.1 μm for mixes containing over 10% SDW was 90% compared to 70% for the reference and mixes containing up to 10% SDW. This may explain why only the reference and the mix containing 10% SDW exhibit considerable attack under both sulphate environments.

Generally, the exposure to magnesium sulphate is assumed to be more aggressive than sodium sulphate because the magnesium sulphate ($MgSO_4$) not only reacts with the CH and C-A-H phases, associated with sodium sulphate attack, but also the C-S-H phases (Matthews, 1995).

The evaluation of C-SDW blends indicated that samples exposed to sodium sulphate were at greater risk than samples exposed to magnesium sulphate. Samples were totally destroyed between 180 and 210 days when exposed to sodium sulphate, whereas, samples exposed to magnesium sulphate, although showing considerable deterioration, were not destroyed.

Although C-SDW blends appear to perform better in magnesium sulphate, they did exhibit a significantly larger weight gain when immersed in magnesium sulphate compared to samples immersed to sodium sulphate. This was attributed to the formation of additional hydration products filling the open pores within the hardened material, as the weight increase did not coincide with increased expansion. The increase in weight could be due to the formation of the white layer on the surface of the samples when exposed to magnesium sulphate, however, this was not observed with C-SDW mortars with SDW contents exceeding 20%.

The evaluation of C-SDW mortars appears to indicate magnesium sulphates promotes strength loss, and sodium sulphate promotes expansion. This was attributed to the type of products formed during hydration and the rate at which these products are formed. Sodium sulphate mainly reacts with the CH and C-A-H to form gypsum and ettringite, which can result in expansion and deterioration through cracking, similar to that observed with the C-SDW blends immersed in sodium sulphate. Magnesium sulphate reacts with the C-S-H as well as the CH and C-A-H to form gypsum, magnesium hydroxide (MH) and magnesium silicate hydrates (M-S-H). The direct attack on the C-S-H results in a softening of the material and strength loss, rather than expansion and cracking.

When exposed to magnesium sulphate, a white layer formed on the surface of C-SDW mortars with SDW contents between 0% and 20%. Hime and Mather (1999) reported that if concrete was exposed to magnesium sulphate ($MgSO_4$), brucite (MH) was formed. The formation of brucite was said to occur at the sample surface, which may enhance sulphate resistance. Bonen and Cohen (1992) reported the formation of a brucite layer on cement pastes cured in magnesium sulphate. A retardation of the sulphate attack was observed and attributed to the low solubility of the brucite layer, which restricted the penetration of the Mg^{2+} into the interior of the sample. For C-SDW mortars with SDW contents in excess of 20%, no white formation was observed. This was attributed to the reduction in calcium hydroxide (CH) formed as a result of the reduction in cement content, which reacts to form CH, and because of the pozzolanic reaction occurring from the large amount of fly ash in the mix, which consumes CH to form additional C-S-H.

26.5 Application in the construction industry

Smith (2006) reported on the management of FGD residues in different regions, including Europe and the United States, and their utilisation and application in sectors such as the construction industry. The key to utilisation of these wastes is finding high-volume applications that are local to the source of production.

At present, there is a vast array of knowledge on the characterisation of desulphurised wastes, but due to their variability, knowledge behind the performance of desulphurised wastes in construction applications is somewhat limited.

In the UK, power stations like DRAX in Yorkshire produce a variety of waste materials as a result of the processes implemented (DRAX, 2015). These wastes include wet-process gypsum, fly ashes, and bed ashes (FBC).

The fly ash, which consists of very fine particles of ash that are removed from the exhaust gases to prevent emission, is used to replace cement, filler, or aggregates in concretes. The furnace bottom ash (FBA), which consists of larger pieces of ash that are removed from the bottom of the boiler, is used as aggregate to make lightweight aggregate blocks. The amount of fly ash sold to the construction industry in 2012 was 750 kt, which is approximately 60% of the total fly ash produced at DRAX, with the remaining 40% still going to landfill. With regards to FBC, 100% of the material produced in 2012, which equates to 290 kt, was sold to the construction industry. With regards to FGD gypsum, 100% of the gypsum produced, approximately 779 kt, was sold to local manufacturing facilities and was used in the manufacture of plasterboard in 2012 (DRAX, 2015).

Other high volume uses of FGD gypsum include gypsum blocks, floor screeds, and plasters where high-quality raw materials are required. The manufacture of plasters normally requires some blending with natural gypsum to improve performance and particle-size distribution (Smith, 2006). The use of high quantities of FGD gypsum can produce products with a variety of properties, including high strength, improved durability, and improved aesthetic properties.

Gypsum produced from the wet process is also used in the manufacture of cement to regulate the setting of the clinker to avoid flash setting (Neville, 1995a), however, this is generally a relatively low-volume usage, approximately 3–6%.

The use of FGD wastes in high-volume cement replacement applications is still relatively novel, however, it has been shown that mixes can develop high strengths and improved physical and durability properties, similar to that observed with fly ash, which is commonly used at volumes of cement replacement between 25% and 50% (Mangat, 1999).

Blending wet FGD gypsum with other materials, such as fly ash, have been reported to improve reactivity, and blends containing up to 70% replacement of cement have been shown to exhibit reasonable strength development (Mangat, 1999; Khatib et al., 2007b, p. 395). However, the binders appear to be more suited for use in improving durability properties, for example, resistance to sulphate attack (Mangat and El-Khatib, 1992).

FGD wastes have also been reported to be a source for the production of recycled lightweight aggregates, which can be used in the concrete of masonry blocks (Smith, 2006; Mangat, 1999). This type of application appears to be suited to materials from semidry and dry processes where constituents of the material in cement-bound systems do not have such strict controls over physical and chemical properties.

Other concrete applications, which have been researched, include the use of FGD wastes in cellular concrete technology to form cellular or foamed concrete (Brylicki and Lagosz, 1997).

The evaluation of FGD wastes for use in flowable fills has been reported (Butalia et al., 2001). Flowable-fill mixtures composed of large amounts of fly ash and water have been used as fills to stabilise poorly compacted soil or granular fill. Issues have been reported in these applications due to large volume changes, however, extensive research has led to its use in numerous applications such as bedding and backfill for utility trenches, paving subbase, bridge abutments, and retaining wall backfill (Flowable Fill, 2015). The benefits of using FGD waste in this type of application stem from the fact that the materials used do not have to meet the quality standards normally associated with concrete and other FGD applications, as well as their use in high volumes.

There are issues associated with FGD wastes that can limit their use in construction applications. Smith (2006) reported that in some instances there are trace elements, such as mercury, which may be present in the FGD wastes. Controls are being put in place to address such issues. However, these types of systems can be quite expensive, so it is likely that many wastes may still be sent to landfills.

26.6 Sustainability of desulphurised (FGD) waste in construction

Sustainability and sustainable development are a common phrase used within society today to assess and/or quantify our impact on the environment, and to enable us to find ways of reduce this impact. As the world's population continues to grow, it is difficult to see, and is often questioned, how human societies can achieve environmental sustainability (Worldwatch, 2013; Sylvia and Doris, 2011).

One area that has had a detrimental impact on the environment is the growing need for energy due to the rapid increase in world population. This has been achieved by the burning of fossil fuels and the use of the coal-fired power stations, which historically, have been a major cause of pollution. The current consensus is that the use of raw materials such as black and brown coal in the production of power will continue worldwide. In reality, global consumption of coal has been continuously increasing (Khatib, 2014), especially in light of recent issues surrounding the implementation of nuclear power, and the efficiency and cost of systems such as solar, wind, and tidal.

Over the last 30 years, FGD equipment has been installed to existing or new power stations to reduce SO_2 emissions, which have played a major part in the pollution of the environment. In the early 1990s, DRAX power station in the UK was one of the first facilities to install wet desulphurisation equipment to control SO_2 emissions. The IEA Clean Coal Centre database had over 900 FGD installations on record in 2005 with a total capacity of 316 GWe, and with further increases planned (Smith, 2006).

The burning of fossil fuels and the cleaning of flue gasses produces wastes, in the form of ashes, insoluble calcium compounds, or a combination of the two. These materials vary in quality, and can contain impurities. Therefore, historically, many wastes have rarely been utilised and have been disposed of in landfill sites. In recent times, this disposal to landfill sites is becoming problematic from an environmental perspective due to a reduction in available sites, and increasing disposal costs. There is also greater awareness over the effects that landfill wastes may have on the environment through leaching of elements into the soil and groundwater (Clarke, 1993).

Therefore, if the current trend is to continue with burning fossil fuels to provide power, then systems need not only to be put in place to reduce their impact in the environment through air quality control, but also to utilise, where practicable, the wastes produced from this process. In 2006, it was reported that there was approximately 45 Mt/year of FGD waste produced in the United States, EU15, and Japan alone, with only 50% of this waste being used in construction applications (Smith, 2006). The volume of FGD wastes from Asia and China were not fully reported, however. Due to economic issues, many power stations do not have FGD processes installed, and therefore the wastes produced and utilised are relatively low with respect to the rest of the world.

Currently, waste materials produced as a result of power production and from FGD systems are used widely throughout the construction industry, especially wet FGD wastes, which are predominantly used in the manufacture of plasterboard. Ash is produced in multiple forms, such as fly ash (PFA) and FBA, which can be used as a replacement for virgin aggregates and cements. The utilisation of waste materials from these types of processes reduces the demand for virgin materials and can also reduce transport costs and the impact of CO_2 emissions.

The key to the utilisation of wastes within the construction industry is the characterisation and classification of different wastes, and developing their use in high-volume applications, which can be manufactured at source to reduce production and transport costs and impact on the environment. For example, most wallboard manufacturing plants are now located in close proximity to the power plant, and tailor their process to use FGD gypsum, which has replaced the need for mined natural gypsum, and no longer requires this type of material to be placed in landfill (Smith, 2006; Plasterboard Sustainability, 2013).

However, in the UK, the construction industry faces a dilemma as the power industry starts to look at alternatives to burning fossil fuels, and improves current installations such as DRAX by converting them to biomass and gas to produce energy. It is expected that by 2016, a third of all UK coal-fired power stations will close so that EU air quality legislation is achieved (Energy UK, 2015).

This may result in a reduction in the volume of ash and gypsum available for use in construction applications, which may once again put natural resources at risk from over extraction.

At present, the UK construction industry annually consumes 420 million tonnes of resources and 6500 hectares of land are converted from rural to urban areas. The extraction, processing, and transportation of these resources cause large emissions of carbon dioxide (CO_2) into the atmosphere and thus contribute to the pollution of the environment (Khatib, 2014). This is why it is important to continually develop methods to manage the utilisation and reuse of natural resources and combine this with recycling of waste products.

26.7 Future trends

Sustainability of desulphurised wastes will hinge on their ability to be used in applications such as the construction industry. However, the push to devolve fossil fuel burning in the production of energy around the world may result in developing countries using more cost-effective FGD systems, semidry and dry processes, rather than more SO_2-efficient systems such as the wet FGD process.

It has been shown that across the world there is a massive variation in material produced from FGD processes, and it is very difficult to assess and predict their performance in cements, aggregates, and concrete systems containing desulphurised wastes.

Therefore, it is very important that work continues to:

1. assess the chemical, physical, and mineralogical properties of a wider range of desulphurised wastes;
2. assess the performance of desulphurised waste on properties of cement-bound materials, with emphasis on their performance in construction applications;
3. assess the performance of simulation of desulphurised wastes to try to identify suitable uses based on physical, chemical, and mineralogical properties;
4. focus on high-volume applications to maximise use and thus reduce the environmental impact of using desulphurised wastes;
5. focus on applications that can be manufactured and utilised close to the source, thus reducing the environmental impact of using desulphurised wastes.

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Conclusion

27

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This chapter attempts to summarise the main conclusions of various chapters in the book in addition to making recommendations and highlighting future trends regarding the sustainability of construction materials covered in the book, in the hope that doing so will minimise the impact of construction materials on the sustainability of our environment.

Chapter 2 concludes that there are still varied approaches with regard to life-cycle analysis (LCA). Traditionally, the LCA for the use of construction materials is based on a 50-year lifespan. The work in the chapter indicates that urgent actions are required to include the production of materials, use of local materials, use of renewable resources and recycling in the LCA. The methodology of LCA requires continuous updating to reflect changes in policy associated with sustainability. In addition to governmental and institutional bodies that set the standard, manufacturers and suppliers should be consulted when drafting the appropriate standards. Eco-labelling may be one way towards achieving the sustainability agenda. There is potential in the future for the LCA to be used for selecting appropriate construction technologies that would have minimal impact on cost, society and the environment.

Chapter 3 deals with the physical properties of materials that affect the durability of their performance and, therefore, their sustainability. These properties include porosity, pore size distribution, permeability and their effect on moisture movement and thermal conductivity. Chapter 3 concludes that greater awareness of these properties can enhance our understanding of and appreciation for assessing the sustainability of construction materials.

The nanotechnology chapter (Chapter 4) suggests that t more research will be done on the use of nanomaterials for construction applications. These studies could help to develop a deeper understanding of interactions between nanomaterials and the original construction materials; studies could include more experimentation and techniques as well as modelling methods. Other areas of investigation could include the effects on the health of people who handle the nano materials as well as their impact on the wider environment. In addition, in order to use nanotechnology in construction, new standards must be developed, or the existing standards need to be modified.

Using glass as a construction material is the subject of Chapter 5. There it is suggested that future utilisation of glass in construction will incorporate two aspects. The first aspect relates to the use of smart and truly responsive glass facades where the properties change to actively control solar gain, daylight, glare and thermal emissions in passive home building. The second aspect relates to the use of glass and glass-hybrid systems as load-bearing structural members, which may include glass panels, roofs, floors, staircases and partitions that may be load-bearing (ie, structural members).

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In addition, waste glass could be used as aggregate in concrete and bituminous mixtures. Also, ground glass may be used as a partial replacement for cement.

Chapter 6 concludes that the demand for metals will be continuous. Actually, the demand is expected to increase, especially in areas where economies are emerging, such as China, India and Brazil, as these countries are aspiring to achieve a high standard of living. New technologies (eg, nanotechnology) will see the utilisation of nanotubes in electronics applications. These new materials are likely to have superior properties (eg, strength) compared to traditional materials. Recycling of metals will continue. However, increasing the life-span of metal components (eg, structural elements) already in service should be given high priority because doing so will be the most commercially viable option. Life span of metals used in structure can be enhanced through the provision of measures to prevent corrosion and choosing appropriate materials such as stainless steel and aluminium alloy.

As indicated in Chapter 7 that many countries in the world are moving towards low-carbon and resource-productive economies, therefore, use of more wood-based products is expected in the future. Also, hybrids of softwood and hardwood will be utilized more because high-performance construction elements can be produced, which may include timber composites and post-tensioned timber frames. Using timber in construction is not only beneficial to the environment (eg, less CO_2 emission) but also can lead to materials that are economically viable.

Chapter 7 highlights the fact that more engineered wood products (EWP) will be used in the construction of buildings and other projects, instead of using only concrete or steel. As indicated in the book’s introduction, many multistory buildings have been constructed with EWP. The advantages of using EWPs are their low-embodied energy, timber’s low level of carbon emissions as well how quickly they can be constructed and the associated cost savings and economic benefits. EWPs will allow the combined use of timber containing a high structural grading with lower-grade timber, thus maximising the overall value of forest resources. It appears that EWP has an exciting future in construction applications.

Chapter 9 is concerned with the sustainability of aggregates in construction. The chapter concludes that the construction industry faces local shortages of aggregates, and this is likely to occur more frequently unless aggregate resources are managed adequately. Such management includes the use of aggregates from alternative resources such as recycled concrete and asphalt pavement, use of low-quality aggregates and waste materials, efficient utilisation of resources by reducing the quantity of aggregates through adequate design and may include underground mining. The chapter stresses that proper management of sustainable aggregate resources is required in order to reduce the impact that the production of aggregates has on the environment and also in order to meet the needs of society.

Chapter 10 deals with the production of structural concrete containing lightweight aggregates made of waste clay produced during tunnelling operations. It is suggested that using this type of lightweight aggregate in the production of structural concrete would lead to (1) a reduction of waste clay disposal from tunnelling and deep construction activities, (2) the production of lightweight aggregate that will reduce the density of structural concrete, (3) a reduction in CO_2 emission, (4) an increase in durability, (5) a reduction in the size of foundations or the construction of taller buildings and (6) the production of lightweight concrete with an adequate in situ compressive strength for structural applications.

Chapter 11 concludes that the use of aerated blocks and cavity walls to increase the insulation will become more popular in the UK and other countries in the world. Because of environmental awareness and the drive to save energy, the regulations will be tightened so that thermally efficient products using masonry will continue. The use of single-leaf walls made out perforated block units will be utilised. Also, non-fired clay blocks using innovative techniques (Chapter 21) will be used more in order to reduce the energy caused by brick firing. It is also suggested that more off-site manufacturing will find more use in the future so as to maintain the quality of products and to reduce waste.

Chapter 12, which deals with the sustainability of natural stone as a construction material, suggests that building with stone can produce structures that are durable and resilient to changing climates. Natural stone does not involve firing as do other materials and therefore emits less CO_2. The development of technical guidance is expected to increase the use of natural stone as a construction material.

Chapter 13 concludes that after a critical analysis of the sustainability of shelled compressed earth block, the material seems to have a great potential for use in construction applications especially in many of the world's developing areas. Using the new technique, shelled compressed earth block will have more adequate mechanical properties in addition to superior weathering resistance as compared to other earth construction materials. The material will have a lower environmental impact and lower costs than other construction materials. Also, more social acceptability of the materials is expected because their appearance is similar to other available materials.

Good practices in the future road of construction are described in Chapter 14, which is concerned with the sustainability of bituminous materials. These practices include the provision of good rideability and safety characteristics, good bond between layers and durability, use of materials from sustainable sources or production methods and the extension of the life-span of road pavements.

As stated in the book's introduction, Chapters 15–17 are concerned with the sustainability of cement-based materials in construction. In brief and in order to produce concrete-based materials that are sustainable, the use of cement replacement materials and recycled products is necessary. Cement replacement materials are necessary, including coal fly ash, ground granulated blastfurnace slag, silica fume, ground glass natural pozzolans or calcined clay (eg, metakaolin). Recycled and waste materials can be used to replace aggregates, materials such as recycled glass, brick and concrete and incineration bottom ash. The use of replacement materials is an efficient way of reducing the burden on the environment because of construction activities. They would reduce the embodied energy and improve the durability of construction materials, thus contributing towards the sustainability of our environment.

Chapter 18 concludes that alkali-activated materials and geopolymers are expected to play an important role in the provision of durable and sustainable alternatives to Portland cement binders. These include environments where the construction materials are subjected to aggressive chemicals and high temperatures. However, like other new materials, field data on the long-term performance of geopolymers as construction materials is lacking, which may restrict their utilisation. With more long-term data, the use of alkali-activated and geopolymer materials is expected to increase.

The chapter suggests that use of vegetable fibres in construction will develop agricultural industries that employ millions of people, especially in the poorest regions of the world. It concludes that the use of vegetable fibres in different applications, including construction, is increasing throughout the world. Vegetable fibres can be used in polymer and cement-based materials, which will provide a non-food source for economic development for farmers. The use of vegetable fibres can be advantageous as it is largely coming from waste and can provide appropriate solutions to housing in rural settings. The building elements produced will have low-embodied energy. However, the durability of construction products incorporating vegetable fibres needs to be investigated further.

Chapter 20 is concerned with the use of fibre-reinforced polymer (FRP) in structural applications as a replacement to steel. The research community plays an important role in achieving this goal, in that it provides guidance for the design and installation of FRP in structural applications as well as producing a sustainable FRP product. Future trends will also include the production of FRP using alternative and more environmentally friendly materials. FRP has been used in strengthening concrete structures, and in the future, it is expected that FRP will be used in the repair and rehabilitation of existing steel infrastructures. The development and adoption of a probabilistic approach in the codes of practice will be required; doing so will allow designers to efficiently use FRP materials and will also enhance the sustainability of structures.

Chapter 21 concludes that if proper design procedures are implemented, the use of fibres in certain concrete applications can contribute to the sustainability of materials. This is referred to as engineered cementitious composites, which can include high-volume fly ash in the mix in conjunction with strain-hardening fibre. Also, in reactive powder concrete, fibre, especially fibre-reinforced plastic, can be used to improve ductility while strength is maintained. Extending the life of existing structures (eg, coastal structures) is possible with the use of fibre-reinforced cementitious composites. Enhancing the service life can include retrofitting and external strengthening of the various elements. The chapter concludes that engineers, contractors, law makers and all those involved in construction should address the sustainability issue when designing a structure. This should include the initial cost of the project as well as the service-life cost.

Chapter 22 deals with the utilisation of wastepaper and wastepaper sludge ash in construction applications. The concept of using wastepaper sludge ash in construction applications is relatively new. However, because natural resources are decreasing, the future will see more utilisation of waste paper and wastepaper sludge ash, which can take the form of cementitious and non-cementitious materials. The use of wastepaper sludge ash as cementitious binders has shown promising results in that the performance was even better than those made with cement.

In Chapter 23, the benefits of using waste rubber in construction is highlighted. Waste rubber can be used as aggregate in concrete and even in bituminous mixtures. It is possible to produce construction elements with good sound absorption properties by using waste rubber. With a carefully designed method, the use of waste rubber in construction can be a sustainable option.

Chapter 24, which deals with the utilisation of sewage sludge in construction, concludes that there is a potential for using raw sewage sludge as water replacement in concrete and cement-based applications. This will have a positive impact on our environment as it will save the much needed fresh water that can be used for human consumption. In addition, the traditional, expensive way to treat wastewater can be altered, which would create more economical benefits. However, more research on the use of raw sewage sludge as water replacement is required in order to have confidence in its utilisation. Given the environmental and economic benefits, using raw sewage sludge as water replacement will contribute to sustainable development.

Chapter 25 concludes that, because of an increase in demand, gypsum mining will also increase. However, demand will force the construction industry to use more synthetic, waste and recycled gypsum. In Europe, it is anticipated that by 2035, 25% of recycled gypsum will be used. It is also possible that new plants will be built in which products comprising 100% of recycled gypsum can be produced. Also, waste gypsum includes products such as flue gas desulphurisation (FGD) gypsum coming from desulphurisation processes. It is expected that the FGD waste will be used more in the future since many countries are opting to use a wet process in the coal power industry where fly ash and gypsum are fully separated.

Chapter 26 deals with the utilisation of desulphurised waste in cement-based applications. It is anticipated that the consumption of coal will continue well into the foreseeable future. Desulphurisation processes are referred to as clean coal technology. Therefore using the waste residues from these processes presents a particular challenge, due in part to their broader compositions. Future trends will include greater utilisation of these products (ie, high volume) in various applications, including construction. Applications include using these wastes in the production of plasterboard, cement and aggregate replacement in various construction applications.

It is hoped that this book will benefit people who use construction materials, including designers, practitioners, policy makers academics and researchers, so that construction materials can be used in a way to minimise the cost of projects and the impact on our society and environment.

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