ICE manual of structural design: buildings

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Foreword

This manual is part of the ICE series aimed at disseminating best practice, and addresses the important field of structural design. There are many very good texts available which provide guidance on detailed design procedures and compliance with codified rules, but the broader issues concerning conceptual design and structural form are rarely treated. Yet these are every bit as important as the detailed aspects – carefully thought out creative design concepts are frequently what will distinguish an outstanding design from one which is merely adequate. And in an era which has seen wealth of structural design excellence, and a healthy increase in demand from clients for high quality structures which do not necessarily fit with conventional design, it is perhaps even more important for us to strive for the best. This manual has a welcome focus on such generic issues and in doing so helps to plug an important gap.

Designers are also facing new challenges, requiring a broader knowledge base in areas such as sustainability and fire engineering, and demands for the structural use of unconventional materials such as glass. These areas are becoming increasingly important, and it is particularly pleasing to see them included here, alongside topics such as risk, controlling computer assisted design, and tolerances; these are too easily glossed over, but of course can be critical in achieving high quality.

I would therefore like to thank the contributors to this manual, all of whom who are at the top of their profession, in assembling such a useful guide which I have no doubt will be of great value, and I recommend it to you.

Professor Roger Plank BSc PhD CEng MICE FIStructE
University of Sheffield
Past President of the Institution of Structural Engineers

Preface

Structural form, structural analysis and structural design often have incompatible requirements and demand decisions that later impact on design, erection and service life. The later a decision is taken, the greater the cost to the client. Consequently, the client must provide well-defined comprehensive requirements and needs for the structure. For example, is the design life related to any of the following: the estimated, expected, intended, predicted, reference, or residual service life; the appearance; any future changes of use or planned functions; the type of structure; space requirements; sustainability requirements? Offering alternative structurally sound designs to the client may be correct, but the client's team must have the knowledge and experience to understand the decisions they are asked to make.

Structural form is developed by considering natural forms using intuition and precedents, and by understanding structural principles, supported by computer and physical models followed by analysis of the structural form. The advantage of using computer software is its ability to allow consideration of many options. Structural analysis is about understanding connectivity, materials, modelling form, structural behaviour and validation of computational output. Good structural design must satisfy architectural, constructional, environmental, and geotechnical material, structural adequacy, and sustainability issues.

Every structure has aesthetics, form and function which require interaction between all stakeholders, with the project being supervised by an appropriately qualified competent person backed by an experienced, knowledgeable, proficient professional team. There must be clarity in responsibilities with a focus on consultation, effective engagement and quality management of all key stakeholders.

Uncertainties affecting structural performance cannot be eliminated and are considered, based on probabilistic concepts of structural reliability and available experience. For example, modern structural design codes provide principles for the design and verification of structures with regard to durability, safety and serviceability, related to the design resistance of a structure at the ultimate limit state and the relevant criteria for the serviceability limit state. Thus the structure must be designed and executed so that it will, during its intended life, with appropriate reliability and in an economic way, sustain all actions and influences likely to occur during execution and use, and remain fit for the purpose for which it is required. This means that two sets of conditions must be considered: (i) the design load does not exceed the design resistance of the structure in the ultimate limit state (associated with structural failure); and (ii) the relevant criteria for the serviceability limit state (related to the structure's use or function) is no longer being met. These limit states are vital as the typical design working life for a structure may be well over 100 years.

This book represents an amalgamation of good, simple rules and principles, and is a repository of best practice guidance in the core areas of civil and structural engineering. It provides a comprehensive reference on structural design for practising engineers, university students of civil and structural engineering, senior engineers requiring information outside their immediate field of expertise, and non-experts in structural design who require reference information.

The book is written and edited by a wide selection of leading specialists in each of the three major areas covered: fundamentals, concept design, and detailed design.

The manual is divided into three sections: (i) Fundamentals of structural design (Chapters 1–6); (ii) Concept design (Chapters 7–16); and (iii) Detailed design (Chapters 17–21).

Fundamentals of structural design

Chapter 1 *The place of the structural engineer in society.* Chris Wise examines the historical purpose and origins of engineering as a professional activity, and how these have helped to develop the role and tasks of the structural engineer in contemporary society. The chapter then describes the mindset required by structural engineers and their future place in society.

Chapter 2 *Tackling structural engineering projects.* Chris Wise explores the underlying patterns in projects and highlights the relationships within them that contribute to successful outcomes. As the changing needs of society require the development of a new genre of engineers, this chapter includes suggestions for the engineer's approach to the future.

Chapter 3 *Managing risk in structural engineering*. Edward Tufton discusses the process of risk management to maximise success and minimise loss. The engineer is very much concerned with durability, safety and serviceability; for the client, this translates into reliable, cost-effective service in support of the building's function. Many codes include allowances to cover commonly experienced risks, however the management of risk involves identifying performance requirements and hazards so that the engineer and the client can appreciate how risks affect the structure's use and how to control them.

Chapter 4 *Sustainability*. Elisabeth Green considers the application of policies and tools on low carbon design of buildings, starting with an overview of global and local policy, moving onto how sustainability can be measured, and then addressing the basic principles of sustainability for buildings throughout the design and construction processes.

Chapter 5 *Taking a through-life perspective in design*. Stuart Matthews considers a through-life perspective to the design process by studying the through-life performance of existing structures to help understand the drivers associated with life-cycle cost, value and sustainability issues. Attention is given to the requirement for a through-life performance plan and a co-ordinated approach to construction, structural design, service life design and associated through-life care processes. Observations are made of future challenges and opportunities which might be expected to influence the design process.

Chapter 6 *Controlling the design process*. Iain A. MacLeod shows how to reduce the incidence of errors in structural design. The strategy is to adopt a questioning approach to inputs and outputs, paying special attention to the validation of models and the verification of the calculated results.

Concept design

Chapter 7 *Key issues for multi-storey buildings*. John Roberts suggests that the success of multi-storey buildings is judged across all design disciplines by the client and the user. The design develops through an ordered series of stages, with excellent communication shared across the whole team. The interrelationship of the different uses within the building must be understood, with direct vertical load paths being strongly preferred. The engineer must also have input into 'non-structural' issues such as cladding, corrosion, fire prevention, partitions, plant and stairs, thus contributing to the success of the building as a whole.

Chapter 8 *Typical design considerations for generic building types*. Gary Rollison informs engineers, who are about to consider a structure for a particular end-use, of the areas of design that they should specifically consider to deliver an appropriate brief for a client.

Chapter 9 *How buildings fail*. Tony Marsh introduces the concept of building failure, covering areas such as progressive collapse and failure due to ongoing serviceability issues, the requirements of adequate pre-construction investigations, building foundations, external environmental issues, material suitability and structural stability. The knowledge and experience of the designer and the checking engineer are also discussed.

Chapter 10 *Loading*. Richard B. Marshall, David Cormie and Mark Lavery categorize different types of loads and the modes of application are explained. The following loads are discussed: permanent (dead), imposed (live), earthquake (seismic), fire, fluid, ground, wind, self-straining (creep, movement, pre-stressing, shrinkage, temperature), silo, soil, and wind.

Chapter 11 *Structural fire engineering design*. Tom Lennon explains the methodology underpinning the structural fire engineering design process which consists of three basic components: (i) choosing an appropriate design fire; (ii) using this information to derive the temperatures of the structural elements; and (iii) assessing the structural behaviour with respect to the derived temperatures.

Chapter 12 *Structural robustness*. David Cormie explains the basis of design for structural robustness, giving practical guidance to designing against disproportionate collapse, and advice on other issues that need to be considered.

Chapter 13 *Soil–structure interaction*. Mohsen Vaziri and Tim Hartlib introduce soil–structure interaction and summarise available methods for predicting the behaviour of foundations and substructural elements constructed within a soil mass. Effective communication between geotechnical and structural engineers is the key.

Chapter 14 *Materials*. David Doran gives advice on concrete, glass, masonry, metals, polymers and timber. Materials of suitable strength, stiffness, flexibility, durability and affordability are key to the realisation of good design. It is also essential that the use of materials is economic and that a high degree of recycling is achieved.

Chapter 15 *Stability*. John Butler discusses how stability is provided for various types of building, and how different actions on buildings give rise to instability – for example, building 'sway' is considered. First- and second-order structural analysis is described, and for each form of construction, different structural arrangements are discussed.

Chapter 16 *Movement and tolerances*. Paulo Silva provides an overview of movement and tolerance issues, and how they affect the work of the practising structural engineer.

Detailed design

Chapter 17 *Design of concrete elements*. Owen Brooker gives an overview of the design of concrete framed buildings and shows how, once a particular system has been selected, the preliminary sizes for both the floors and the columns are determined.

Chapter 18 *Steelwork*. John Rushton outlines an approach to steel design from the viewpoint of a consulting designer and presents summaries, design guidance and sources of information on the design and construction of steelwork to achieve a successful outcome.

Chapter 19 *Timber and wood-based products*. Peter Steer discusses the design of structural timber components and fasteners, describing the basic properties of wood and wood-based products, identifying their characteristics in environmental conditions such as humidity, temperature, and duration of load.

Chapter 20 *Masonry*. Andrew Rolf provides background information on the basic concepts of masonry construction and its components, covering load-bearing and non-load-bearing construction techniques. The chapter also provides rules of thumb for initial sizing and provides introductory information on the design of masonry under seismic conditions.

Chapter 21 *Glass*. Mauro Overend provides a unified method for the structural design of glass elements in buildings, and advice on material selection and connection design based on the limit state design philosophy.

Structural design can often be perceived as following codes, checking standards, and using computer software. The range of topics covered in this manual show that imagination, flair and teamwork are also prerequisites. I would like to thank the authors, writing chapters in their particular area of expertise, for their diligence, and say how much I have learnt from them.

Professor John W. Bull

Acknowledgements

During the editing of the manual, I have been extremely fortunate in being helped and guided by many people. I would like to thank Victoria Thompson and Matthew Lane, the Associate Commissioning Editors at ICE Publishing, together with Geoff Taylor and Ian May for their expert assistance with reviewing material. Many other acknowledgements are given in the chapters and are too many to mention individually; I am very grateful for their contributions.

A most difficult task was assembling the content, and for this I am indebted to Peter Waldron, Ian May, Chris Playle and John Carpenter who carried out this important commission with the foresight to ensure the superb coverage in the manual.

I am also indebted to Roger Plank, past president of the Institution of Structural Engineers, for agreeing to write the foreword.

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Chapter 1

The place of the structural engineer in society

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An examination of the historical purpose and origins of engineering as a professional activity, developing into the contextual role of the engineer today. Looking beyond problem solving and the application of theory to a consideration of the role and tasks of the structural engineer in contemporary society. Contains a review of the relevance and traction of the stated aims of the professional educational system. Constructive criticism of these requirements in the context of the needs of twenty-first century society and changing demographics of technology, ageing, globalisation and sustainability. Describes the kind of mindset a structural engineer needs to develop and the future place of structural engineers in society.

CONTENTS

1.1 Our challenge

Over thousands of years, the craft of designing and building elegant and safe structures has remained a core challenge for post-nomadic human society. For millennia, our structures were provided by those who gained knowledge and skill from their elders and by studying precedent, by following lore and rules of thumb, and by learning on the job, by experience, failure, success and feedback. The last 100 years of professional structural engineering have seen this ad hoc approach regularised with the development of safeguarded minimum standards, so that overt structural failure has become less and less common. But in the entirely reasonable attempt to regulate out the dangerous, we have also seen a disconnection between structural engineers and the people we serve, coupled with the broader development of disciplinary protectionism and, given the money to be made, the emergence of some powerful vested interests.

Today we have a raft of locked-in rules and behaviours. Many are useful, but we have many sacred cows that we defend even though, regrettably, we can no longer remember their justification. Take safety factors, office floor loading and deflection limits, to name but three. Nowadays, the professional structural world is big business, and it has a huge inertia. Yet the delicate challenge we face has never been more human as the world develops, and we find ourselves needing to do more, for more people, with fewer resources. So it is a challenge to describe the core knowledge a structural engineer might need today and in the future. There may not be a single answer, but each engineer, be they diligent analyst, careful experimenter, inquisitive thinker or free spirit, needs to find his or her own understanding, with the ultimate goal not of competence but of mastery. Only through mastery will we be able to move the world forward. This is a task that requires great and continuing study and effort.

Unfortunately, I certainly cannot say that if you learn this fact, or this theorem, this code or this stratagem, you will become a good structural engineer. But I can say that without a basic knowledge of engineering science and its application you may end up designing unsafe structures.

In addition, in order to maximise the profession's chance of making a meaningful contribution to the world as structural engineers we will need to concentrate as much on behaviour and context, the natural environment and societal changes as on technical wizardry.

This is not the first time I have tried to capture this precious thing called knowledge. While teaching a few years ago at London's Imperial College with my partner Ed McCann, we dreamt up a list of the sort of things an engineer might want to know, and our list is shown updated in **Table 1.1**. You may be surprised to see there is little about the sizing of beams, but quite a lot about behaviour, awareness and attitude. On closer inspection, you will see that very little of this knowledge is subject to formal education or formal professional training, but nevertheless for our sustainability I believe we need it. There are at least six areas which are outlined in **Table 1.1**.

1.2 Context

To begin at the beginning: What is the need to which today's structural engineer is the answer?

It is traditional to say we need structural engineers because 'they stop things falling down' or, tellingly rarely but perhaps more positively, 'they keep things standing up'.

Or how about the structural engineer's role as skilled problem solver, 'they do for a penny what any fool can do for a pound'…but that could equally apply to a painter/decorator.

The activity of structural engineering is actually rather important. It underpins human history, and is one of the things that separates us from the apes. Everybody does it, whether they call themselves an engineer or not. Of course some people are better at engineering than others, and some of us even do it professionally. Structural engineering is one of the oldest professions…in every town, in every country, in every era the ubiquitous structural engineer is there, diligently sizing the beams and testing out the foundations in response to natural

1. Being tuned in to the context	**2. Taking relationships seriously**
Understanding the nature of engineering: What is it?	Developing a personal approach to the art of engineering
Dealing with natural forces	Presenting your work effectively
Understanding the societal need for engineering	Developing a sense of ethics and values
Understanding engineering's stakeholders	Getting the best from a group of differing personalities
Understanding precedents for failure and success	Team working
Mastering the language of engineering and making it accessible to lay-people	Designing for sustainability
	Confronting vested interests
3. Developing judgement through projects	Respecting the environment
Understanding the stages of a project	How the design process works in a team
Understanding the key drivers of a project	Linking knowledge across subject areas
Understanding the site	Being persuasive
Understanding the brief	
Understanding the iterative nature of design	**4. Respecting the place of theory**
Planning and being part of an effective team	Statics
Balancing skills across a project	Dynamics
Planning and hitting deadlines	Geotechnics
Getting properly paid	Hydraulics
Understanding how to manage costs	Maths and statistics
	Stability and equilibrium
5. Mastering technique	Serviceability and failure
Understanding how to use computers wisely	
Free-hand sketching in 2D and 3D	**6. Integrating construction with designing**
Understanding scale and natural proportion	Making time for research and prototypes
Technical drawing and the use of CAD	Understanding how to build things
Assessing loads and their uncertainties	Understanding how to use tools
Respect for the properties of materials	Designing for manufacture
Building structural analysis models	Designing for construction
Applying the thought process of structural analysis to other natural phenomena	Designing to be safe
Using 'What if's'	Understanding the sequence of construction
Using rules of thumb and empirical rules	Working to a construction-driven programme
How to use design codes	
How to tackle open problems	
Understanding subjective and objective tests	
Understanding how to make judgements	
Getting more from less: Integrating structure and environment	

Table 1.1 Six key behaviours for structural engineers

forces. It is reasonable to assume one would prefer a building to be built on rock rather than on sand!

For every house built on sand there are works of great brilliance too. Jacob Bronowski in the documentary series *The Ascent of Man* gives the example of a gothic cathedral in which not only has mankind managed to think of stone as a building material, but has also made the tools with which to dig it out of the ground and then to work it. Not content with that, the cathedral maker has also decided to stack the stones, one on the other, to make an immense enclosure, perfectly balanced, totally in tune with the natural forces which act upon it. But the cathedral maker's defining achievement, in doing all that, is in managing to make something that transcends the inanimate stone-ness of the stone and goes on to move the spirit. Somewhere in that process, the engineering became art…we call it architecture. To create such an emotional connection from a pile of stones is a remarkable achievement, and the structural engineer's knowledge and skill is at the heart of it.

There is a lot of great architecture that is driven by engineers: think of the Millau Viaduct, the Sydney Opera House, the Skylon and the Colosseum. We are the latest in the line that produced such great work. Yet I am often told by engineers, 'Oh yes, we'd love to do such things but we never get the chance.' This gives rise to a basic question about the making of opportunity in response to a need, similar to the choice faced by early humans as the population grew beyond the natural capacity of the land to sustain them. They could starve, they could move, or they could change the environment. Almost uniquely in the animal kingdom, human beings do not always have to adapt to suit the environment…in many cases they are able to adapt the environment to suit them. So of course, early humans took the decision to re-engineer the nature of wheat by domesticating it to give a higher yield, and the rest is history. Here we are thousands of years later as proof. In this little analogy lies the key to our own usefulness, and our very sustainability. If we are able to 'read' our environment and respond to it by design we will have a role in life. This question can be addressed to all aspects of our work from natural, through social, to personal and business.

The biblical story about the foolish builder who built his house on sand still rings true, encapsulating our daily challenges as structural engineers very neatly. So that old knowledge is still useful. Yet is our structural engineering for the twenty-first century the same as it was a millennium, a century, a decade, a year or even an hour ago? We had better understand that first, as only then can we think about the sort of core knowledge such a structural engineer might need.

1.3 What structural engineering is

As a clue, we can look at the UK's first engineering institution, the Institution of Civil Engineers (ICE). Their original 1828 Charter defines Civil Engineering as:

> *the art of directing the great sources of power in Nature for the use and convenience of man, as the means of production and of traffic in states both for external and internal trade, as applied in the construction of roads, bridges, aqueducts, canals, river navigation and docks, for internal intercourse and exchange, and in the construction of ports, harbours, moles, breakwaters and lighthouses, and in the art of navigation by artificial power for the purposes of commerce, and in the construction and adaptation of machinery, and in the drainage of cities and towns.*

The compass of this statement may once have been enormous, but it seems very limited today. This is because it was written at the start of the age of steam and railways. But the first bit is good, and still appropriate. While teaching at Imperial in the early noughties, Ed McCann and I re-wrote the broad definition of what we do as:

> *Engineering…the art and practice of changing the physical world for the use and benefit of mankind.*

Now the use of the word 'art' in this definition (and the original) begs the question: 'Is engineering an art, or a science?' Many engineers say 'science'. Certainly many politicians say 'science'. But others have clear views, for example:

> *Here, indeed, is the crux of all arguments about the nature of the education that an engineer requires. Necessary as the analytical tools of science and mathematics certainly are, more important is the development in students and neophyte engineers of sound judgment and an intuitive sense of fitness and adequacy.*
>
> *No matter how vigorously a 'science' of design may be pushed, the successful design of real things in a contingent world will always be based more on* ***art*** *than on science. Unquantifiable judgments and choices are the elements that determine the way that a design comes together. Engineering design is simply that kind of process. It always has been, it always will be.*
>
> Ferguson (1994, p. 193–194)

In 1978, philosopher Carl Mitcham observed that:

> *Invention causes things to come into existence from ideas, makes world conform to thought; whereas Science by deriving ideas from observation, makes thought conform to existence.*
>
> Mitcham (1978, p. 244)

He could have substituted the word *engineering* for *invention*. In considering the essential distinguishing nature of engineering it is helpful to know that:

> *Science begins out in the world and ends up inside your head.*
> *Engineering begins in your head and ends up in the world.*
> *In that sense, engineering is the exact opposite of science.*

I agree with those who believe that the practice of engineering is an art, which makes substantial use of theoretical knowledge derived from science. Therefore, we could capture the essence of structural engineering simply by putting a qualifier on exactly what part of the physical world we are concerned with. When we do so, we obtain something that, when tested, stands up to scrutiny and so becomes a useful piece of knowledge:

> *Structural engineering: The art and practice of changing the structure of the physical world for the use and benefit of mankind.*

1.4 The changing way we work

The structural engineer was originally an artisan craftsman – 30 000 years ago the sort of person who could take a collection of tree branches and mud and fashion them into an effective shelter. Even for a simple project like that, such a person would know something about previous attempts, what worked before, about local materials and their availability, tools and their use, construction planning, the use of human labour, team-work, transport, durability, maintenance, in fact much of the knowledge we need nowadays. Some of these people became clearly highly skilled and practical, and they eventually achieved great things. For example, around 2150 BC they managed to quarry 'foreign' blue stones weighing 4 tonnes each in the Preseli mountains in south-west Wales and move them probably on rollers and rafts through 240 miles of rolling and sometimes wooded countryside and across rivers before tilting them up into place to make the first Stonehenge. A century and a half later their descendants pulled the 50 tonne Sarsen stones from the Marlborough Downs 25 miles away, dressed them and raised them into position, all without the use of iron. Even well-known modern engineers have found this process surprisingly tricky, no doubt protesting in their embarrassment that the folk-knowledge that underpins such feats was lost about 3999 years ago.

The structural engineer is the inheritor of a mantle described around 45 BC by Vitruvius as the provider of at least the first two parts part of the mantra of 'firmitas, utilitas, venustas': or 'firmness, commodity and delight'. In Roman times the engineer was often a military man who wore the uniform of a legionary but was actually a member of the 'immune' class. These immunes possessed specialised skills, qualifying them to perform duties atypical of a Roman soldier. As immunes, engineers were also clumped in with other such worthies as musicians, artillerymen, weapons instructors, military police, carpenters, hunters and medics. Being 'immune' exempted them from the more tedious and dangerous tasks other soldiers were required to do, such as ditch digging, and also entitled them to receive better pay. The immunes weren't entirely untouchable – they still had to go into battle *in extremis* so they needed to keep their fighting skills up.

The Roman engineer was expected to be able to turn his hand to the construction of everything from watercourses to roofs to ships to giant catapults. I know from personal experience trying to build one of their 23 tonne torsion spring ballistas out of oak and imitation antelope tendons that the Roman engineers managed to push their simple natural materials to levels of performance well beyond those we can justify using modern codified methods. The lesson for our code-dominated age is that codes are only a catch-all approximation and often inhibit structural performance. From Abyssinia to Greece to Rome to medieval times the engineer and his successor the Master Builder developed their knowledge largely on the basis of trial and error.

Of course we do not see evidence of many of the errors, as the fallen masonry and its engineers are long gone, but there were many disasters. Perhaps the classic is Beauvais Cathedral. It still stands proudly, if wonkily, in northern France, with half of its body chopped off. According to research by Jacques Heyman of Cambridge University, in 1284 the nave of what was at 48 m tall the tallest cathedral church ever built collapsed like a pack of cards when a statue that was preloading part of the pinnacle and flying buttress system fell off, perhaps hit by lightning. At that point one small part of the thrust line in the compression-based system moved outside the geometry of the stone section, and down it fell, removing in turn the support for each of its neighbouring vaults and eventually the whole nave. Beauvais contains another juicy lesson for the structural engineer: No matter how confident you are in your structural system, it is always good to have Plan B.

No doubt all of these engineers, from Stonehenge on, were also practising the art of crisis management from time to time as they tried to stop structure X from collapsing into the water, or roof Y from landing on the heads of the assembled worshippers. Often the response to failure took the form of simply repairing, and usually enlarging, the part that had failed and each person saying they were not responsible for its design and construction. The Great Fire of London in 1666 heralded a rethink about safe building practice, and a rise in masonry buildings in lieu of the combustible timber houses. Soon after, thanks to Isaac Newton and the Newtonian mechanics we are still taught at school, it became possible to apply some theory to the subject, and even to offer some tentative predictions of structural behaviour. *Force* = *mass* × *acceleration* is a very useful thing to know, but these assessments often looked at the performance of a structure under just one natural phenomenon, usually gravity.

Even the great Thomas Telford relied on a lot more than this early engineering science. In his biography (2002), Samuel Smiles records how, after the raising of the chains on his trail-blazing gravity-defying 570 ft span Menai Bridge in 1826, 'the great man was discovered by his friends down on his knees in grateful prayer'. Telford attributed the rocking and rolling of the bridge to the wind gusts bouncing back up from the ground and pushing the suspended deck upward (an effect we now know to be induced by vortex shedding of the sharp edges of the bridge). Later, just before 1 September 1857, Isambard Kingdom Brunel tested the strength of his Saltash Bridge by loading the great 455 ft long tied arches on the bank of the River Tamar with 1000 tonnes of ballast. He measured the downward movement under this great weight – it moved down only a few inches and British Rail (now Network Rail) still have the results drawings to prove it…so all was right with the world and Brunel pronounced the spans safe to lift into place. The fact that for the century and a half since then the Saltash Bridge has been subject to rolling train loads which behave quite differently to static ballast and impose unforeseen bending and unforgiving fatigue loads on the structural system meant that later engineers had to strengthen Brunel's structure with additional cross-bracing and stiffening. So what is undoubtedly a great structure and one of my favourites was achieved by a combination of some rudimentary structural analysis (in modern terms) coupled with trial and error and the wisdom that comes from a study of precedent – and a century and a half of structural sticking plaster. As I know only too well from the Millennium Bridge, one thinks one has covered every single load case in your predictions at one's peril.

Our use of engineering materials has changed, but very slowly. For millennia human beings used stone, timber, mud and bricks, fibre ropes and fabric. In the late eighteenth century, at Iron Bridge in Shropshire there was the first use of cast iron as a serious building material. For much of the nineteenth century engineers used wrought iron and rivets – limited to half tonne billets which translates to lots and lots of tiny iron sheets and enormous labour demands to join them all together. The late nineteenth century saw the introduction of the rolled steel beam, and in the early twentieth century engineers adopted reinforced concrete and then pre-stressed concrete, 2000 years after the Romans first used concrete most notably for the shell roof of the Pantheon in Rome. Then slightly purer mathematical shells and membranes and timber grid shells were used. Since then, not many new materials have been introduced, except perhaps structural glass and cable nets, which are not desperately inventive for a multi-billion pound industry and faintly embarrassing when measured against other fields. Compared to aviation, computing, medicine, communications and other technological industries, the basic world of structural engineering has changed little since the building of the Empire State Building in 1930, all 57 000 tonnes of it in six months. We have got slower, although hopefully a lot safer, which shows where we have chosen to place our values.

In fact, a look at the technological and economic waves of change (**Figure 1.1**[1]) shows us that most of the key developments over the past century have been in areas other than structural engineering. There have been five waves since the industrial revolution. But every technological wave reaches a crest and then dies back after its products become a commodity, a point we reached in traditional structural engineering some time ago. We are just beginning the sixth wave, entering what has been described as a Century of Biology.

Why do we need to know this for the engineering of structures? Well, the social environment whose needs we serve is changing inexorably and ever more quickly. For example, the UK's two main waking activities now are working (only three hours), and leisure (five hours and rising) (UK Office of National Statistics), in stark contrast with the situation 100 years ago. At the same time our population is getting older (see **Figure 1.2**) and this will have a direct effect on our own careers as structural engineers. As the UK pension pot is sadly not sustainable, the inevitable consequence for most of us will be careers well into our 70s and beyond. Šmihula (2009, p.3247) observed that the technological waves get faster and faster, with the next (genetics/robotics) wave only expected to last 25 years or so, and the next even less. So, for the first time in technological history, we are witnessing waves of technological change that are much shorter than our engineering careers, meaning that the context in which a 'structural' engineer operates will inevitably change during the life of an individual, and probably more than once.

Of course, even with its mighty compass, structural engineering is just a small subset of all engineering. As a matter of fact we often argue, in a jolly sort of way, that architecture is also just a subset of engineering and in terms of the definition of engineering given earlier in this chapter, it is. Understood correctly, engineering has a great and unfettered range, and its power is such that engineering skill gained in one area, such as structures, should be transferable to another as the technological waves roll along. This ought to be a good thing, but of course demands pro-active behaviour from its cohort, not something the structural world has shown itself to be very adept at. We occupy a world in which the engineering profession in the United Kingdom is regulated by ECUK through 36 engineering institutions, licensed to put suitably qualified members on the ECUK's Register of Engineers. These titles are protected by the Engineering Council's Royal Charter. Structural engineers are but one of these 36 disciplines. Like all protected species, engineers are at risk when their habitat changes, Royal Charter or not.

Engineering in practice, even structural engineering, is not a static activity. Instead it is in a state of continual change,

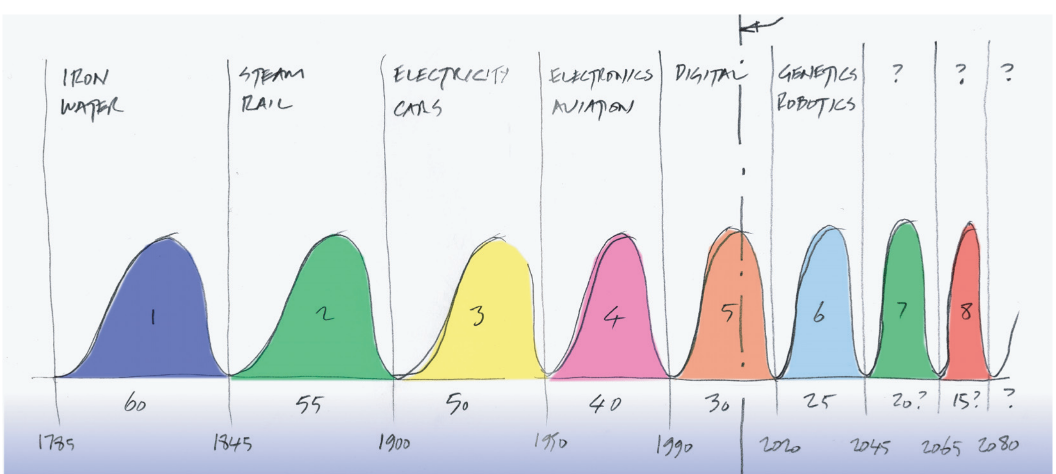

Figure 1.1 Technological waves are getting shorter

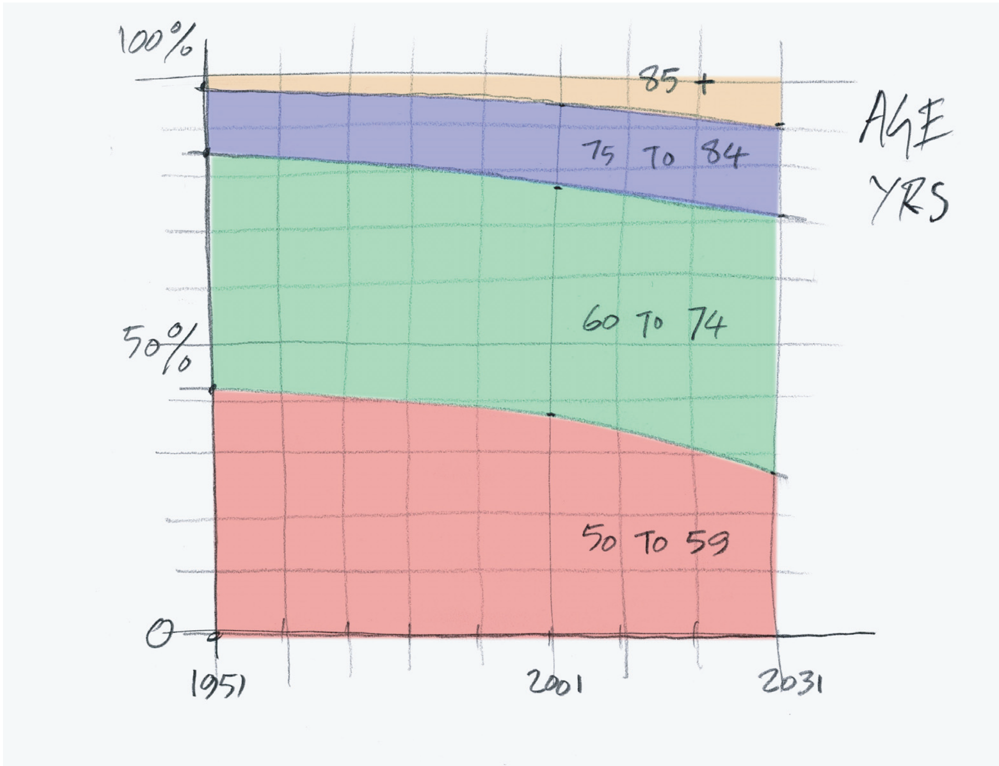

Figure 1.2 We are getting older (data taken from UK Office of National Statistics)

ideally existing in a happily dynamic equilibrium with its environment, or to be explicit, in equilibrium with the needs of society within *its* environment. To be a successful engineer with a satisfied life it is important to be able to distinguish between useful long-term skills and knowledge, and yet be able to pay attention to the changing short-term 'noise' in the system. Recent examples of 'noise' that spring to mind are Progressive Collapse (after the Ronan Point disaster in 1967); Ultimate Limit State Design in the 1970s; Quality Control in the 1980s; Concurrent Working (after Sir John Egan) in the 1990s; Sustainability and Climate Change in the 2000s. This is not to say that these are unimportant, quite the opposite, but more to acknowledge the need to be tuned to continuing change in the dominant uncertainty of the moment, dominant uncertainty being the Thing that makes you wake in a cold sweat at night.

Of course once a dominant uncertainty is tamed, a new one rears up in its place. A forward-thinking engineer needs antennae to spot it, and an entrepreneurial spirit to respond to it. Isambard Kingdom Brunel and Robert Stephenson were great entrepreneurs, and it was this rather than their technical skill that gave them the possibility to design and build the engineering projects for which they are celebrated. Their example is a further answer to those who lament 'no one ever asks us to do interesting stuff'.

There are contemporary examples of entrepreneurial engineering such as Arup's championing the re-routing and eventual design of the Channel Tunnel Rail Link, but not so many in the purely structural world where design codes are very prescriptive.

1.5 The balance between theory and practice

A theoretical knowledge of engineering science is important, and is well covered elsewhere in this book. But it is revealing to know how much a part theory plays in the characteristics required of a structural engineer by its professional institution. The Institution of Structural Engineers, in its 'Notes to Candidates (for Chartered Membership)' (http://www.istructe.org), says that Chartered Members of the Institution will be able to demonstrate:

- a sound understanding of core structural engineering principles;
- the ability to use relevant existing technology coupled with the ability to locate and use new research and development to benefit their work and structural engineering generally;
- the ability to solve complex structural engineering problems and produce viable structural design solutions using appropriate methods of analysis;
- the ability to exercise independent judgment in the application of structural engineering science and knowledge;

- technical, management and leadership skills to plan, manage and direct human, material and financial resources;
- commitment to the public interest in all aspects of their work, including health, safety, risk, financial, commercial, legal, environmental, social, energy conservation and sustainability;
- effective communication and interpersonal skills;
- knowledge of the statutory and other regulations affecting current practice in structural engineering;
- a significant base of information technology skills;
- commitment to the profession of structural engineering, particularly with regard to the Institution's Code of Conduct and the requirement for Continuing Professional Development.

Arguably only two of these ten characteristics are formally theoretical (they are shown in italics). The rest depend on context, relationships, technique, projects and construction, and give an insight into the limited role of engineering theory in the practical life of a twenty-first century structural engineer. This is not the same situation as, for example, just after the Second World War when the likes of shell mathematicians Ronald Jenkins and Felix Candela, geotechnical engineer Alec Skempton and their contemporaries justifiably prided themselves and their profession on their ability to do 'hard sums' by hand, with minimal artificial aid.

My view is not new. Fifty years ago, the MIT Committee on Engineering Design said:

> *The designing engineer who remains on the frontiers of engineering finds himself making only a small fraction of his decisions on the basis of numerical analysis. When the problem becomes older and more decisions are based on numbers, he moves on to a new and more difficult field where he again finds that a small fraction of his decisions are based on the kind of analysis taught in engineering schools. This is not to try and belittle the importance of analysis. Everyone recognizes it as an essential tool of the trained engineer. It does not, however, answer all or even a majority of the questions an engineer must answer in a typical design problem, particularly a new one.*
>
> *It seems unlikely that numerical analysis will ever answer more than a small proportion of these questions. The remainder must be decided on the basis of ad hoc experiment, experience (the art of applying knowledge gained by former experiment on the same or similar problems), logical reasoning and personal preference. The subconscious reasoning process, which we call intuition, can play a large part.*
>
> MIT Committee on Engineering Design (1961, p. 650)

A few years ago we explored this in a review of student performance in our third year group design projects at Imperial College, London. These students are now growing up in practice and approaching or are beyond professional chartership. At the time, there was little correlation between their assessed ability to perform our quasi-realistic design projects and their ability to do other more theoretical subjects (see **Figure 1.3**). This led to a tentative conclusion that if they are any good at design it isn't because they are transferring skills from their theoretical course but because of things they have learned elsewhere. This supports the view that the preoccupation with engineering science is somewhat misaligned as preparation for today's mainstream engineering practice. From this we concluded that to improve the ability of the students to carry out real engineering projects the curriculum should be redesigned so that relevant non-theoretical skills were properly taught and rewarded.

Discussions with the academic staff and experience in design classes showed that most students could not cross-link

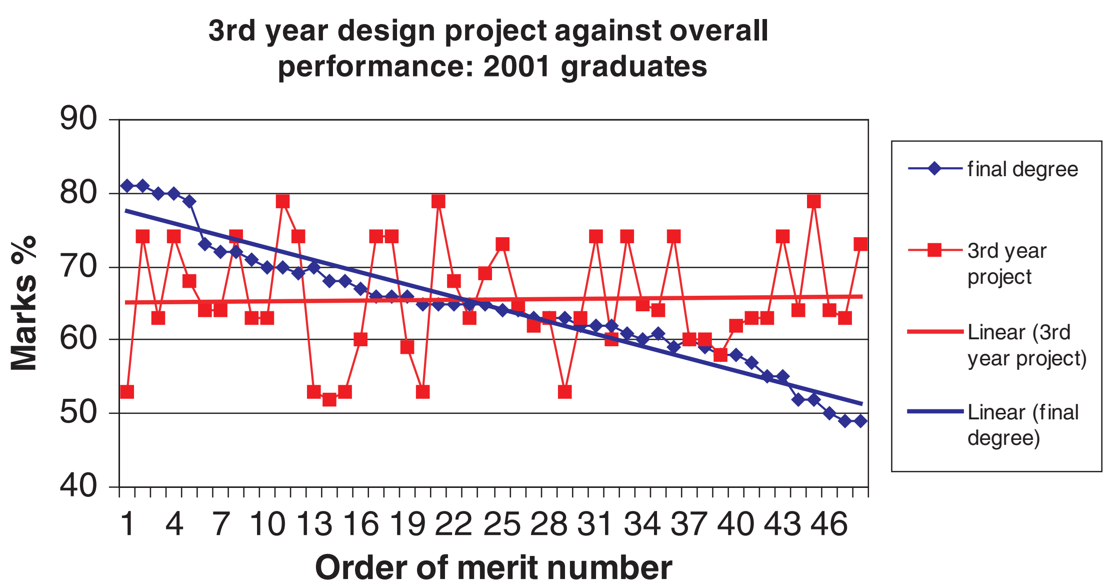

Figure 1.3 Typical lack of correlation between design project and overall course marks

their theoretical knowledge so that it was useful or even see how it would ever be of practical use. They exhibited a collective failure to apply the wide range of engineering science that they were learning to real and messy engineering problems. When they enter practice we see this translated into inhibited behaviour by professional structural engineers that is often:

- technically knowledgeable within carefully defined boundaries, but
- strongly subservient when it comes to conceptual thinking, and
- contextually challenged.

As an insight into industry's thoughts about this, in 2005 the Royal Academy of Engineering commissioned a study from Henley Management College, covering four main areas:

- changes in the industry;
- current and future skills requirements;
- the comparative quality of UK and international engineering graduates;
- consequential requirements for changes in engineering degree courses.

To quote the study:

> *Companies identified Information and Communications Technology (ICT) and Materials as key areas for increased graduate recruitment to support future growth. Although industry is generally satisfied with the current quality of graduate engineers* ***it regards the ability to apply theoretical knowledge to real industrial problems as the single most desirable attribute in new recruits****. But this ability has become rarer in recent years – a factor which is seen as impacting on business growth.*
>
> *In descending order of importance the relevant attributes for graduate engineering recruits identified by industry include practical application, theoretical understanding, creativity and innovation, team working, technical breadth and business skills.*
>
> Royal Academy of Engineering (2007, p. 7)

There is evidence here that industry's need for engineers who can design real engineering projects conflicts with the 'closed' nature of much theoretical teaching as real projects are almost always 'open' problems, involving the transfer and application of knowledge from everywhere. As engineers, it is projects, not facts, that we are commissioned to supply, and which provide our means of expression. The results of the study are starkly revealing and support our earlier findings at Imperial.

1.6 The engineer's relationship to key stakeholders

Structural engineering is something done by humans, for humans. Humans live in a complex ecosystem on planet Earth, and we depend on it for our long-term survival. In the 140 pages of *Gaia: A New Look at Life on Earth* James Lovelock pioneered the idea of the Earth as a complete living system, the largest of all known living and breathing creatures (Lovelock, 2000). I am a strong believer that we are people first, engineers second, and structural engineers third. And we have obligations to all of these stakeholders and to the planet that supports us.

To most people in the developed world at least, the dominant uncertainty of the modern world is not what happens to structural engineers. The dominant uncertainty of our time is survival on this planet. In response, engineers must stand up and say 'We helped get the world into this mess, and we are going to make it our mission to get it out again.'

We spent many years working with engineering undergraduates at Imperial College, and they contain a significant number who know they really do want to save the world. As I have said, their mission is to exercise the art and practice of changing the physical world for the use and benefit of mankind.

So, to the stakeholders (**Table 1.2**), from the general to the specific. In terms of need, I have used the word 'should' instead of 'do', for this is an aspirational list of achievements. The structural engineer's relationship with the complex needs of these stakeholders is not primarily through theoretical knowledge, or the doing of hard sums, or the science of materials, or the analysis of problems, or expertise with spreadsheets, although all of these play some part. The main way in which an engineer engages with the world is through projects, working with the public and with many people from other disciplines, and the evolving approach to projects is the subject of the next chapter (Chapter 2: *Tackling structural engineering projects*).

1.7 Conclusion

This chapter has examined the historical purpose and origins of engineering as a professional activity, developing the contextual role of the engineer today and going beyond problem solving and the application of theory into the consideration of the role and tasks of the structural engineer in modern-day society. A review of the relevance and traction of the stated aims of the professional educational system has been given with a constructive criticism of these requirements in the context of the needs of twenty-first century society and the changing demographics of technology, ageing, globalization and sustainability. Lastly it has described the sort of mindset a structural engineer needs to develop and the future place of structural engineers in that society.

Stakeholder	What need should we be answering?	What happens if we fail?
Everyone on Earth	■ The need to keep things in the same physical and functional relationship to each other under the actions of natural forces	■ Schools collapse; hospitals topple; offices crash; walls come tumbling down
Everyone again	■ The need to use the Earth's resources wisely	■ Some go without now ■ Others go without later, perhaps very close to home ■ Wars over energy and water, fossil fuels, and maybe renewables
Any country	■ The need to generate a successful part of the country's creative economy, a key plank of any county's global trading platform ■ The need to be proud of our generation through our achievements	■ High unemployment ■ A gradual relative decline in living standards ■ If we don't continue to invest, we'll lose our place at technology's global top table ■ The country loses out in the global competitive environment for technology, with major new investments going elsewhere
Any public sector	■ Need to use taxpayers' money wisely	■ State support for technological and non-technological projects will be cut
Private sector	■ The need to use technology wisely to meet business needs, with all of the positive ripple effects that enables, through such things as employment generation, income and trading enabling, without compromising the well-being of others	■ Our cities and communities will become fragmented by self-interest groups and physical artefacts
Academia	■ The need to arm up the next generation of engineers with a contemporary skill set ■ The need to do research that is directly relevant to the part and practice of contemporary engineering ■ The need to think independently and develop techniques that challenge vested interests (see the example below)	■ We will teach people to do things that were important once but were long ago automated ■ We will miss developing the knowledge and skills people need for their own and everyone else's survival
Industrial conglomerates	■ The need to develop new technologies to conserve resources	■ High embodied carbon ■ High first cost (narrower investment)
Ourselves	■ The need to broaden the use of our skills of numeracy, literacy and creativity to meet the world's big challenges including resource depletion, climate change, global poverty, population growth and international security	■ Someone else will
Our children and their children	■ The need to provide for the needs of the present without compromising the needs of the future	■ Persistent reduction in the standard of living

Table 1.2 What can structural engineers do for their stakeholders?

1.8 Note

[1] Please note the artwork for these chapters feature the author's hand drawings as would be done in practice during the design stage.

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Chapter 2

Tackling structural engineering projects

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Projects are complex and varied, and require many different approaches. This chapter explores some of the underlying patterns in projects, and highlights the relationships within them that can contribute to successful outcomes. Given the longevity and apparent maturity of the structural world, it might be expected that the profession has reached a common position about the skills and techniques that are useful, but the changing needs of society require reflection and, if necessary, development of a new genre of engineers. The chapter contains some material designed to tease out personal preferences towards working methods, and includes some suggestions for the engineer's approach to the future.

CONTENTS

2.1 What is a 'project'?

There will always be projects – they are the vehicle by which we exercise our art and modify the structure of the physical world (for the use and benefit of mankind of course). As time goes by, the nature of the projects will change, and so will the tools and techniques we apply to them, but they remain the lingua franca of structural engineers.

Figure 2.1[1] is a key diagram that anchors the intellectual activities at the heart of every *project*. The process is eventually linear, but experiences many overlapping feedback loops along the way in the search for the best solution. Every engineering project arises in response to a given societal *need*. The engineer and the rest of the design team create a *conceptual response* to the formal expression of the need which is expressed by the client in the *brief*. Then there is the art of critical *testing* and *judgement* that are part of the engineer's work before we can know that a good *'design'* (noun) is produced. Only when that judgement has been made can we confidently go ahead and *make* what has been designed.

The best *'design'* (verb) is an iterative process – it would take a genius to get the right answer first time. *Conceive: test:*

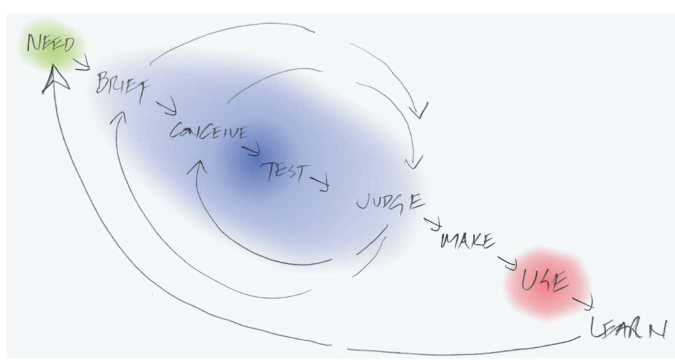

Figure 2.1 Design at the heart of a project

judge, then iterate. That is design. The brief itself cannot help but be woven into the design process, as it is often hard for the client to capture a clear expression of the need first time. Logically, the brief needs to be an iterative project in itself.

You can go further with this diagram and use it at both macro and micro scales. Each and every part of a project can be treated as its own little project, with a similar set of activities, so the anchor diagram has little piggy-back projects emerging at every stage. It is good to know that there is the opportunity for creative conceptual thinking even in the meat of a project. For example, there is conceptual complexity in the design of a temporary works scheme, or perhaps fine judgement required to choose the detailed proposition for a joint between two pieces of steelwork. I once remember spending a day with an architect trying to work out a way to join together two tubes on a steel staircase. We eventually took inspiration from the way a cylinder head is bolted onto the engine block of a 1930s racing car and came up with what we will always think of as our 'Bugatti detail'.

I have been asked many times to teach people how to be better at conceiving ideas, but can only offer the advice that a good concept arises from your life experience integrated with the immediate project context. So to be better at conceptual work it is well worth trying to develop your skills of judgement and cultivate your experience to help you avoid the obvious elephant traps that are not technical but more broadly contextual within the whole project. This will help you avoid the sort of sad projects which don't work very well despite getting every single technical sum absolutely and pointlessly right.

In the context of projects, it is possible to speculate about which parts of the project process are dear to our structural hearts, so here are four hypotheses.

2.1.1 Hypothesis 1: In structural engineering projects (and other sorts of engineering), specialist testing is reasonably well handled

We can't seem to get away from the fact that structural engineers love their special tests. Here we are helped by 3000 years of science and now by computers which are very good at conducting specific tests, especially for closed problems. But to a computer a formula is just a formula; while we may have chosen to separate the world into 'disciplines', this is a matter of supreme irrelevance to a computer. A computer is simply a specialist, specialist in crunching numbers, whether or not those numbers come from the field of structures, fluids, chemistry, maths, genetics, aviation, statistics, climate change or currant buns. By using them only on structures we miss the chance of challenging both the computer and its operator, so we should look for ways to extend that reach.

2.1.2 Hypothesis 2: In engineering projects, success is usually characterised by effective briefing, successful conception and judgement and not just successful testing…and failures follow the opposite pattern

Arguably many failures are not failures of individual tests, but failures to conceive a complete set of tests. For example, the Millennium Bridge has sometimes been quoted as an example of a failure of a mathematical test, but the maths were essentially correct. And the judgements we made based on an extensive suite of those mathematical tests were also correct. But what was marginally less than perfect was our generalist understanding of every single one of the interacting complexities which the suite of tests were designed to address. So, our judgement to go ahead and build was, as sure as night follows day, also marginally less than perfect. The moral of this particular cutting edge story is: 'Please don't believe you can test your way to the answer.'

2.1.3 Hypothesis 3: To raise our game in projects, it is necessary to dramatically improve our skill in conception and judgement

Our friendly computer doesn't care if it does daft tests for idiotic questions…nor does it yet know what to test, or how to develop a concept, or how to make the ultimate balancing judgement as to whether the concept meets its given need. In typical engineering projects, I could speculate that the proportion of design effort might be (say) 3% Conception: 94% Specialist Testing: 2% Judgement. I think that balance is fundamentally wrong. We need big help, so we could do a trade: if engineers are well placed to teach others about testing, others might be well placed to teach engineers about conception or judgement – let's say artists for conception, lawyers for judgement and doctors for the use of precedent. Here at Expedition, for example, we run life drawing classes and classes in visual aesthetics. This sets a test for those engineers who exhibit Myers Briggs tendencies of introversion. Please get out there and find out what other people are doing – some of it will be really useful.

2.1.4 Hypothesis 4: In making a structural engineering contribution to projects, we are good at objective tests, but poor at subjective tests

Objective tests are repeatable and their results will be agreed by everyone. An example of an objective test is 2 + 2 = ? (answer 4). Another example is the midspan beam moment M = ? (answer $\omega L^2/8$).

Subjective tests depend on the observer and their experience. Each person conducting such a test will do it according to their own value set. The results may be different for everyone, or there may be a consensus. An example of a subjective test is: what shall we make the structural frame out of? (answer: it depends). Another example from the Millennium Bridge: is it better to spend money on protecting the lugworms on the bed of the Thames or on providing a disabled lift? (answer: we probably need both).

2.2 Side presumption (the old chestnut): 'University is the best place to teach testing (in the virtual, non-physical world)'

It is said that theory is well handled at university, and practice is well handled outside, and that this is the best fit. The tacit assumption is that conception and judgement are being adequately provided elsewhere, but while university is a good venue for learning about scientific and mathematical tests, it remains a very poor venue for physical and spiritual tests, and non-quantitative judgement.

In response, long ago we conceived the Constructionarium residential construction week for student engineers, with Imperial College and John Doyle Construction, and this event showed us that major educational change was possible, but would only succeed when the design and construction professions made a significant contribution back into academia. For a multi-billion pound global industry, this participation is still rare. The Constructionarium teaches people skills, programming, financial control as well as an understanding of the physical world. Such practical knowledge is a core part of all engineering, and we can thank our predecessors everywhere for all of their efforts perfecting the techniques we have so far needed. It would be a shame not to use them because we are stuck inside our artificial silos.

So what about the theory and technique of conception, and the theory and technique of judgement – who teaches us those? Is there any place in the structural engineer's lexicon for the application of judgement, or are we simply going to be the technical hand-maiden at the service of the ideas of others, whether good or bad? Personally, I know that the more I focus purely on objective tests, the more hand-maidenly I become.

2.3 Relationships

The structural engineer does not work in a vacuum. Every project involves many people whose values and skills are different to our own. The relationships are human, and like politics, marriage or friendship they go through many stages. Like these relationships, they are built on mutual need mapped to self-interest. They go through the first date, the marriage, the honeymoon, family moments, boredom and sometimes fall-out over money or some perceived social slight. At the end there's a legacy and someone benefits, we hope.

2.4 The relationship between structural engineer and client

The concept of a client turns out to be a minefield. Who in fact is your client? There are at least three possible answers to this simple question. It could be the person who commissions you and pays your bills, and this is certainly the traditional belief. 'The client is always right' is an oft-quoted mantra with which I personally disagree because I've seen too many examples where the client is spectacularly wrong. The influences of good and bad clients are given in **Table 2.1**.

Secondly, your effective client could be the person who recommends you for a commission. Most often if you are a structural engineer, this is the architect. The master–servant relationship between architect and structural engineer works very well for some, but of course it can also be exploited, for example, when an architect is performing badly, compromising the project. So often, nothing is said, and bad practice tacitly supported. The only way to bring that to the attention of the commissioning client is effectively to tell tales. This will mean that your architectural 'master' may take offence at the behaviour of the 'servant' and not recommend you for the next project, a position of power which leads to exploitation by the perpetrator and compliant submission and quiet stewing by the victim.

Thirdly, the ultimate client may be neither commissioner nor recommender, but actually the people for whom you are designing, namely the end-user, as part of wider society. Of course, the end-user pays for the project in the end, either in rent, or through taxes, or through tickets so this is not as far-fetched as it might seem at first. If you ask, 'who is a project for?' rather than, 'who is paying for it short term?', you will get this third answer, and that is the one I usually prefer. I think of the traditional paying client as a short-term means to the end of providing the user with something wonderful.

2.5 The relationship between structural engineer and architect

Most structural engineers spend their working lives on building design, in projects led by an architect. During Hi-tech, as in gothic times, the structure was central to the core expression of contemporary architecture. Now Hi-tech has lost much of its power, as social and environmental drivers have assumed centre stage in the search for sustainable answers. Nevertheless, architects are central to the lives of many structural engineers, especially in large pieces of archi-structure such as stadia, airports, railway stations, exhibition halls and pedestrian bridges. The relationship has enormous potential for great and satisfying work. But if communication or mutual respect is poor for some reason, the participants may find themselves condemned to a lifetime of repetitive drudgery, in a form of master–servant relationship, so it is incumbent on the engineer and the architect to speak and to listen to each other. **Table 2.2** outlines the influence of the good and bad architect.

2.6 The relationship between the structural engineer, the environmental engineer and the emerging environmental protocols

Still waiting for the quintessential built example of their collaboration, this relationship is becoming all the time more important, primarily because of changes in environmental legislation and the increasingly purposeful role of authorities such as the Environment Agency. **Table 2.3** outlines the influence of the good and bad environmental engineer. Between them, structure and environment are responsible for about 75% of the cost of most building projects. But curiously with that responsibility comes a lack of control by those who have responsibility for the overall project. In academia, I have been amazed to hear architectural lecturers state that 'Architectural students don't have time to study all of that environmental stuff', which begs the question of how structure and environment can be

The good client	The bad client	The typical consequence of the bad client
■ knows what it wants, or is prepared to explore what this is with the design team while paying them properly for their help	■ does not know what it wants	■ client procrastinates and becomes unable to sign-off design stages
■ communicates its wishes clearly	■ does not or cannot communicate clearly especially with technical disciplines who might expose their shortcomings	■ cannot tell the difference between a good engineer and a bad one and, as a result, treats them all the same both intellectually and in terms of design fees
■ makes intelligent decisions based upon the advice given	■ makes decisions first and looks for justification afterwards ■ unmakes decisions again and again as more information becomes understood	■ unbalanced project decisions as design team is forced to fit its solution into a poorly conceived pot. This is particularly bad where a naive initial cost plan becomes the principal instrument by which design proposals are judged
■ respects the value of consultants' advice	■ has little or no idea what its consultants are trying to do for it	■ client exhibits fight or flight behaviour at meetings
■ is open-minded and able to listen to advice especially in areas away from its core business expertise	■ doesn't welcome advice and therefore perpetuates out-of-date practices and behaviours	■ hinders technological development as client exhibits risk-averse behaviour. Classic examples of the result are BCO offices, and 'Wimpey' homes
■ treats its project collaborators as equals	■ treats its project collaborators as servants	■ expects to issue instructions and have them followed without question, whether rational or not
■ chooses a balanced project team with good chemistry	■ chooses a named and perhaps talented, opinionated and expensive architect, who may then be surrounded with less talented consultants who have to ensure the architect's designs can be built	■ too much power vested in architect. In today's technologically challenging environment, this often leads to wilful, extravagant designs, rather than sustainable ones ■ rewards compliant engineers ■ penalises independent-minded engineers
■ chooses a team on the basis of quality	■ chooses a team on the basis of price	■ design team spend more time protecting their financial position than designing a really appropriate project
■ chooses a project team after shortlisting on the basis of experience and recommendation, or by limited competition	■ chooses a team by open design competition	■ wastes design resources doing required early stage design work on losing competitions. Often there are 100 unpaid teams going for a single commission ■ industry-wide design fees increase, or salaries reduce
■ pays fairly and on time, commensurate with the value of the professional advice	■ pays very little at the start of a project even though it is here that really good ideas are worth their weight in gold and instead expects the design team to subsidise the early stages of the project itself (especially before planning consent is granted) ■ will not pay for redesign work caused by the performance failings, or by the actions of one strong member of the team acting unilaterally	■ reinforces the notion that you can buy engineering advice as a commodity by the pound, or by the can of beans ■ rewards those who do the minimum ■ rewards those who keep quiet about poor design ■ makes it hard for emerging practices to compete as they are unable to subsidise unpaid early stage work
■ working in a field which increases the overall benefit to society	■ is trying to maximise short-term financial profit without worrying about the future	■ best designers seek commissions from the richest clients at the expense of the rest of society
■ practises what it preaches	■ says one thing publicly and does the opposite in private	■ greenwash: many public clients preach sustainability but make decisions on a capital cost basis, ignoring whole-life costs and consequences
■ asks for professional insurance appropriate to the risk	■ cannot be bothered and asks for blanket insurance regardless of project size, commission value, or design risk	■ smaller projects are over-insured; larger projects are under-insured; every member of design team carries same level of insurance regardless of their specialism; overall, design fees higher than they need to be
■ wants the useful fruits of its projects to be used by others	■ places a blanket of commercial and intellectual property protectionism over its projects	■ the wheel is continually being re-invented

Table 2.1 The influence of the good and the bad client

The good architect	The bad architect	The typical consequence of the bad architect
■ has a balanced view of firmness, commodity and delight	■ thinks that architectural 'art' is more important than architectural technology	■ architectural complacency perhaps forgetting the laws of physics
■ lets nature take its course, through the knowledgeable application of engineering thinking	■ always asks for things to be as small as possible	■ sub-optimal designs with poor use of material resources ■ high embodied carbon
■ recognises their own limitations and welcomes the input of others	■ behaves as the 'supreme leader' and tries, perhaps to the client to say s/he understands all aspects of contemporary project design	■ weak architecture due to lack of knowledge
■ is a collaborator	■ believes all should follow her/his directions without question	■ engineers must be compliant and perform engineering design without designing out the underlying problem
■ thinks that environmental performance is as important as aesthetics	■ thinks that aesthetics are more important than performance requirements	■ global warming
■ understands value in more than pure cost terms	■ reduces everything to money	■ only considers the immediate project cost plan when taking decisions

Table 2.2 The influence of the good and the bad architect

The good environmental engineer	The bad environmental engineer	The typical consequence of the bad environmental engineer
■ treats all environmental phenomena the same, as an interrelated set of natural forces	■ defaults to pipes and duct systems	■ we are still building 1960s projects
■ responds to the specifics of the site	■ has a one size fits all answer…'the London solution'	■ we are still building 1960s projects
■ looks at ways to shape the project in response to the orientation of the project and the site	■ treats all elevations the same	■ we are still building 1960s projects
■ chooses environmental systems on a whole life basis	■ chooses plant based on plant room space-take and capital cost	■ overdesign
■ considers the building as a technological and natural whole	■ designs each sub-system to the optimum without regard for the others	■ sub-optimal behaviour of the whole
■ takes expectation and activity into account when setting performance criteria	■ assumes everyone is seated at a desk in T-shirt and shorts.	■ over constraint of the occupied environment
■ considers the façade as a site-specific environmental moderator and the environmental design as a single holistic entity	■ follows legislation	■ west face too hot, south face too bright, north face too cold, east face too much glare
■ looks at the project in its context for sun, wind, rain, taking account of its surroundings	■ looks at the project as if it stands in splendid isolation with nothing around it	■ fails to take neighbourly advantage of borrowed shading, environmental wind protection, overshadowing, reduction in environmental building loads

Table 2.3 The influence of the good and the bad environmental engineer

carefully woven into a building project if that particular architectural degree course remains in the pre-1970s.

Nevertheless, it is a fact that you will not get planning consent for anything other than small projects if you have not carried out an Environmental Impact Assessment. The structural engineer's role is not the most beautiful, normally taking the form of a damage-limitation exercise. The tacit assumption is that as long as the project doesn't make the environment any worse, it will be acceptable. So, at a formal level, the structural engineer is charged with demonstrating, for example, that the groundwater regime will not be adversely affected by the construction of a deep basement; that the construction of a tall building will not adversely affect the wind environment for people walking at street level; that the construction process itself will be as quiet and non-disruptive as possible.

For buildings, there is an emerging requirement through protocols such as BREEAM and, in the US, LEED, to show that the overall construction has reasonable environmental performance (see Chapter 4: *Sustainability* for more details). We can see that this extends naturally into a single whole-life assessment, in which such things as initial embodied energy and eventual reuse potential feature explicitly (see Chapter 5: *Taking a through-life perspective in design* for more details).

There is much work going on to draw the various strands together, and we can expect these will be linked into a single coherent approach. At which point there will be a commercial imperative to deal effectively with the physical relationship between the structure and the environment, within a holistic architectural proposition, from cradle to grave. Structural engineers will need to develop a much broader contextual awareness if they are to play a meaningful part in this new order.

To my mind this relationship is key to the evolution of all three professions going forward, a close-knit group of structure, environment and architecture, although we need to develop a much better mutual understanding. I can see in the not too distant future a time when they become one profession with a series of internal specialisms. Arise, the building technologist.

2.7 Technique

Good technique comes first on a map of your own capability to your aspirations – no matter how much you want it, it's no use trying to be Fred Astaire or Ginger Rogers if you have two left feet. But although we may have been trained for a few years as structural engineers and so are apparently different from those who have trained as shoe-designers, at heart we are all humans and we all have brains, arms and legs, and our experience of life unites us much more than it separates us.

A surprising amount of this life experience fits very well to the knowledge needed by a structural engineer. Of course we have some specialist knowledge which is occasionally welcomed – how to work out the bending at the root of a cantilever, for example. But we also develop skills that enable us to handle many other situations for which we haven't been explicitly trained, at least since early childhood. Perhaps one of the most useful is what we might call 'getting your own way'. This is a raft of tactics and techniques that we use to circumvent opposition so that in the end our proposition sees the light of day. This example is important to structural engineers who are regularly confronted by naivety on the part of those such as clients and architects to whom we are formally and contractually beholden. Knowing how to get your own way without boring everyone or imposing a loss of face on the part of the uninformed is a key part of the structural engineer's persona. This is because the consequences of unintelligent structural engineering are at best damaging to the environment in terms of embodied energy, or because they are downright ugly. At worst, letting others ignore structural advice can be dangerous and even life-threatening.

Structural engineers have historically occupied a spectrum from the bridge hero grappling with waves, wind, dynamics, fatigues, fracture mechanics and complex erection (but possibly not aesthetics), to those who enable dreams to be fulfilled, like the talented archi-structural engineer Peter Rice. Most engineers sit somewhere in the middle and it is this area that has most to fear as it may well largely disappear. Routine work is already threatened by a combination of cheap engineering labour elsewhere, and even that work will soon become automated. Nothing lasts forever. Look at Hi-tech architecture, once the sexiest thing on the planet and characterised by the marriage of architect and structural engineer, vibrant for 40 years from say 1960–2000…but now no more. I can see this reflected in our nominally structural office when we get one person who wants to go to a tall building conference, but 10 who want to attend a conference on green buildings or infrastructure. Our engineers are voting with their feet, and choosing some sorts of knowledge over others, by thinking fundamentally about what they hold to be of value.

2.7.1 What engineers are 1: The Artist, the Artisan, and the Philosopher (2007)

As an engineering designer, how do you work? It appears there is more than one answer to this question, and while running the RSA's Royal Designers Summer School a few years ago we developed some understanding of the various approaches. Observing the behaviour of a broad spectrum group of designers of items from scarves to cars to towers, theatre designer Tim O'Brien and environmental engineer Ed McCann derived a suite of designer personality traits which together seemed to cover everyone they were studying. Reflecting on this, they identified three key generic personality types within the projects, each defined by the way they approached their work. Structural engineers, also being designers, fit into this gallery, and you can take the simple test in **Table 2.4** to find out which type of designer you are. The answers for each personality type are towards the end of the chapter.

There are three possibilities: characterised by O'Brien and McCann as the Artist, the Artisan and the Philosopher. First, the 'Artist', motivated by interest, who finds it easy to start, but hard to stop. Their projects, as represented in **Figure 2.2**, do not follow a straight line. An Artist will change direction often as something captures his or her interest. He is not precious about today's position, is full of ideas, and often ends up somewhere unexpected. There is risk and pleasure in equal measure in working with an Artist.

Next, the 'Artisan', who seeks perfection of form, but can't begin without a pre-existing concept, and then incrementally seeks to improve it. The Artisan doesn't like to go into the unknown, so their projects (as shown in **Figure 2.3**) are rooted strongly in precedent, or codes. You are likely to get exactly what you asked for with an Artisan, so they are loved by well-established commercial sectors.

Last but not least, the 'Philosopher', who seeks meaning, and needs to work very hard to get perfection of the proposition. They attempt to understand all the key project information before beginning anything at all. Philosophers find it terribly hard to start and agonise about changing circumstances, having invested so much effort in perfecting the original meaning. Once they eventually start, they expect the outcome to have great intrinsic integrity. They can be stubborn if asked to change course, and need to feel in command of the intellectual part of the project process. **Figure 2.4** shows a Philosopher's view of a project.

Please answer A, B or C to each question Question	A	B	C
You are walking down the high street when you come across a pocket watch on the pavement. Do you:	Stop and look at it, and wonder if its owner misses it	Step on it	Pick it up and shake it to see if you can fix it
You have been invited to a fancy dress party. Would you:	Dress normally because people are more important than clothes	Spend the 3 days beforehand trying out everything in the wardrobe, then wear a sheet	Hire the costume your friend wore last year
You are walking on a desert island and a coconut lands on your head. Do you:	Pick it up and decide what to do with it in good time, after due reflection	Throw it into the sea, then dive in after it	Open it with the pen-knife you always carry in your rucksack, eat the flesh and drink the juice
Your hosepipe in your garden has become knotted on the reel. Do you:	Go to the shops and buy another one because this is cost and time effective	Prick holes in it	Spend the afternoon untangling it
A friend is in big trouble and asks you to lend her £1000. Do you:	Explore whether she really needs the money or would be better off with another approach	Lend her £10 and suggest she goes to the races	Check your statement and write a cheque agreeing a reasonable rate of interest and a repayment schedule
You are invited to be the first person on Mars. Do you say:	What is the point of space exploration?	Yes, and may I bring my hat?	No thanks, I'll let someone else go first
You are only allowed to take one book on holiday. Do you take:	*Oxford Dictionary of Quotations*	A ball	*SAS Guide to Survival*
There are 3 of you floating in a life raft and there is only enough food and water for two. Do you:	Carefully evaluate who would make the greatest contribution to civilisation on their return…and then brief the third concisely on how to achieve that outcome	Rock the boat	Eat the bloke who went to the party wearing a sheet
It is raining, and you discover you have a hole in your shoe. Do you:	Ask yourself whether it's worth making the journey	Walk right through the puddles and try to get the water inside your socks to warm up	Hop
You are holding a dinner party and you find you have lost your recipe book. Do you:	Phone for a take-away and ask them to deliver 1 course every hour so there's time to talk	Use whatever is in the fridge. What is a recipe book anyway?	Go round to your next door neighbour and borrow their recipe book

Table 2.4 Engineering personality test (for answers see Table 2.7 towards the end of this chapter)

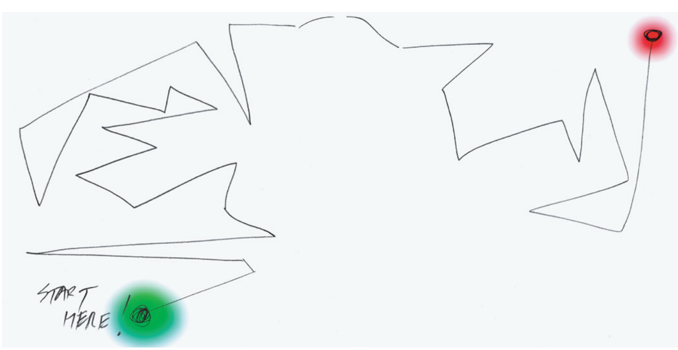

Figure 2.2 An Artist's view of a project (it's an adventure!)

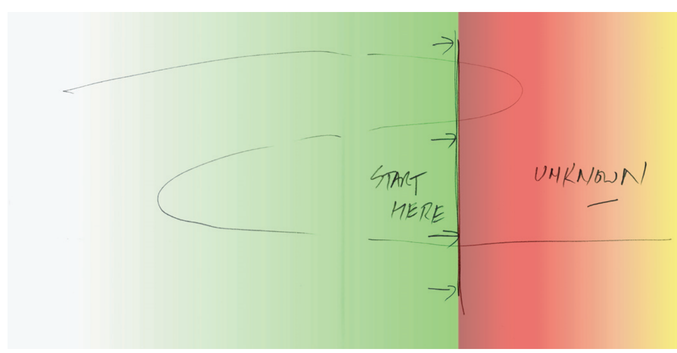

Figure 2.3 An Artisan's view of a project (it's on familiar territory)

Testing this with various audiences around the country we have found that engineers are mostly Artisans, with few Philosophers and Artists. I hope you took the test, in which case you may recognise yourself as one of these, or perhaps a combination.

With that in mind, it is possible to take this light-hearted but effective assessment a little further and draw some conclusions in relation to the way in which particular approaches interact with the key anchor diagram for the project process shown earlier.

Figure 2.4 A Philosopher's view of a project (as a perfect entity)

The Artist is often comfortable at the conceptual stage of the project, and will often start without reading the brief, which is regarded at best as a constraint to work towards later. The Artist is also happy to make judgements about the overall fit of a concept to the brief, especially favouring subjective tests. They tend to take objective tests as 'given', and are rather dismissive of them. They are very likely to try to change the rules if they don't fit a favourite concept.

The Philosopher likes stories (non-fiction). They are very happy working with the Client attempting to understand the Need to which the project is the answer. The Philosopher also enjoys the process of distillation that leads to the production of a brief which fully encapsulates the meaning of the project. The Philosopher is also happy to make judgements about the fit of the tested concept to the brief, but likes to go through a systematic process in which cause and effect are well-linked. They are reluctant to reshape the core purpose of the project too many times as, for them, it is extremely gruelling.

The Artisan is very happy being given a brief and a concept and being told to 'get on with it'. The Artisan is particularly at home with objective tests which have been perfected. They like absolute rules and probably won't challenge them. When dealing with subjective tests an Artisan will try to find a pre-existing example that is close to the suggested proposition and gets very uncomfortable if there is no precedent. An Artisan is unlikely to question the brief. Artisans are very valuable and efficient when it comes to the production stages of projects, where they can use tried and tested systems.

How do these three types relate to each other in a real engineering project? Well, you need an Artist for the concept, an Artisan for the tests, and a Philosopher for judgement (which is mapped back to the brief they themselves wrote). In constituting a project team it is worth knowing that a project generally needs all three types to be successful.

2.7.2 What engineers are 2: the Specialist Specialist

I fully accept that there are many fine Artisans who can test the pips out of a jellyfish and they are worth their weight in gold. Some of the most popular people in our practice are specialists like this: Olympic standard numerical tool builders – people with a civil or structural engineering degree who are able to make us a numerical tool to study raindrops falling on bus-shelters, predict the path of daylight down into an underground station, or show the pattern of moss growth on buildings. They are called 'Specialist Specialists' because they specialise in being a specialist, and delight in it. Arguably we are all trained to be Specialist Specialists at university. The Specialist Specialist has lots of depth, but not much breadth. For this reason, it is rare for Specialist Specialists to do a project on their own.

2.7.3 What engineers are 3: the Specialist Generalist

In contrast, the 'Specialist Generalist' is someone who specialises in being a generalist, and we are a bit short of these. In temperament they are often Philosophers or Artists. I am probably one of these. So is a friend of mine who trained as a civil engineer. For many years he'd never met an architect (or an environmental engineer)…and he was very happy. He worked direct to his client, and as a result his experience grew so he had a broad world view and lots of linkages. He knew he would be called to be responsible for everything on a project, and to think broadly. My friend took an overarching view of a project, and would be familiar to Vitruvius, the medieval Master Builders, and Brunel. The archetypal Specialist Generalist was Pliny the Elder, the Roman scholar. Pliny wrote books on cavalry tactics, biography, a history of Rome, a study of the Roman campaigns in Germany (20 books), grammar, rhetoric, contemporary history (31 books), and his most famous work, *Historia Naturalis* (*Natural History*), published in AD 77. *Natural History* consists of 37 books including all that the Romans knew about the natural world in the fields of cosmology, astronomy, geography, zoology, botany, mineralogy, medicine, metallurgy and agriculture. It just shows how much the human mind can take in (and, as already said, engineers are still human).

In dealing with such complex issues as the response to climate change, the Specialists need the Generalists, and vice versa. On a project, and in education and the cultivation of experience, it is all just a question of balance, according to the Need to which we are expected to be the answer. **Figure 2.5** shows how different types of engineers relate to planet Earth.

2.8 It is within our power to redesign ourselves

I know structural engineers need to be better understood. But each time I say this, I have to remember that many of our clients have little knowledge of our profession, and recognise that we are the ones who have to take responsibility to sort this out. Beyond one or two sentences, we cannot communicate with each other. Perversely, the more infallible we become, the better our projects work and the more invisible we become. We and the society in which we sit are a common species divided by two languages, our technical language and society's non-technical language. Structural engineers need to learn from those who do speak well about the built environment, namely architects, or those who speak well about any other product, namely advertising and branding consultants.

We have every right to be proud of what we have achieved so far. But a comparison between the V&A Museum's 'Unseen Hand' exhibition of 100 Years of Structures with (say) the Boat Show, or the Motor Show, or any electronics' fair, shows that

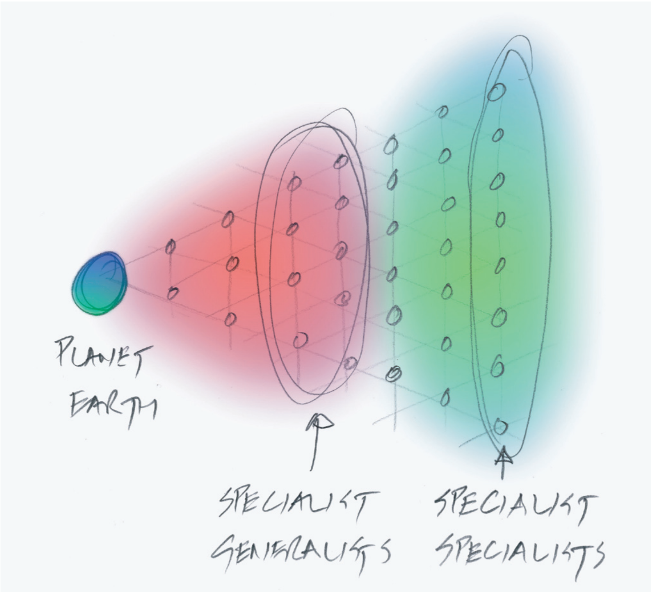

Figure 2.5 How different sorts of engineers relate to planet Earth

structural engineers are also very timid as an industry. We accepted relegation in the V&A (Victoria and Albert Museum) to a very small architectural ante-room. As the dominant questions change from structure (will it fall down?) to environment (can we globally afford it?) and perhaps to society (does it benefit mankind?), so engineering as a way to solve these problems should take centre stage. But the introverted 'specialist specialist' world of pure structural engineering has not yet caught up, and this may partly explain why those outside it appear not to recognise the significance of our achievements.

Nevertheless, the talent is there to turn it around. There is apocryphal evidence to support this, as I have heard environmental engineers say 'The intellectual horsepower of an average structural engineer appears to be twice that of an average environmental engineer.' Many structural engineers harness that horsepower to be superb Artisans, numerically gifted people who love logic and mathematical modelling, but that is such a powerful skill that it needs to get out of its one small corner of the technological world and make a difference. To do that, we need to know how to put our skill into a broader societal context.

Think-tanks signal the change in direction: for example, the 250-year-old RSA (Arts, Manufactures and Commerce) has swung its aim squarely away from the technological waves of design and industry and onto society and environment. The RSA's Chief Executive Matthew Taylor (ex Institute for Public Policy Research (IPPR) and 10 Downing Street policy chief) writes a blog on the RSA's website on the need for what he calls a change to 'Pro-Social Behaviour'. This supports the view that society is the ultimate client for engineers. But Taylor is the first to admit that he isn't quite sure how to achieve it. This is where a generation of specialist generalists – structural engineers familiar with thinking through complex design and testing processes with their many variables and feedback loops – could deliver real value.

In public policy, such as health, education, law and order, complex situations are often tackled with pilot projects but this is costly and often doesn't scale up. I would love the structural engineer to grasp the opportunity and turn those same skills of logic, numeracy and understanding of natural forces towards solving societal issues of even greater interest and even more potential satisfaction. I think my friend, the daylight-mapping-spreadsheet-guru (one such broad spectrum engineer), would be just as able to design tools for social policy as for buildings, if only he properly cultivated his contextual knowledge. As traditional structural calculation heroics become automated, it might be very satisfying for him to do that; and indeed, he might not become extinct.

I know that the structural engineering world probably will not or cannot change itself fast enough, so for this to happen the business and social environmental pressures will have to increase fundamentally. And that will only happen if they have a pressing need which we can answer. Sadly, there will have to be something in it for business, and for politics, be it money, power or comfort, or maybe even survival.

2.9 Getting more from less: Integrating structure and environment

If I think of the most satisfying and successful projects I've been involved with over the past 20 years or so, they share a common theme: they use the structure to manipulate the environment for the benefit of the architecture. Sometimes this is on a macro scale: for example, the great 90 m span concrete shells making up the simple thermally stable backdrop to the old warbirds at the American Air Museum at Duxford, or the vierendeel monocoque used for the whole of the exoskeleton of the Commerzbank tower in Frankfurt to open up giant holes to bring daylight and air into the centre of the building. On these successful projects it was very unusual to work on even a simple thing, say a beam, without integrating its structural behaviour with the natural phenomena with which it comes into contact.

As just one example, which shows the depth to which it is necessary to go, consider just a few of the factors against which a simple structure might be tested by the engineer, and as a result reshaped. Let's consider the most basic structural form – a simple beam (see **Table 2.5**).

This beam is just an easy example, but something every single structural engineer needs to know how to do. The design of a beam is the structural engineer's equivalent of practising scales on the piano – a simple form laden with depth and subtlety, from which you can go on to make much more sophisticated and complex pieces of work confident in the knowledge that you have some mastery of technique. For every simple beam, there is a floor, for every floor a building, for every building a community, for every community a city or a country

Environmental factor	Response in structural beam form
The need to bring in maximum daylight	■ Taper down the beam at the ends near the windows
The need to put the material in the right place and conserve resources	■ Design the 'perfect beam': deeper and narrower in the centre, wider and shallower at its ends
The need for thermal mass to balance out day and night cooling loads	■ Provide a beam system with at least 75 mm of concrete ceiling that can be 'seen' by the interior of the space
The need to take advantage of the free cooling offered by lower air temperature at night	■ Use a beam with ventilated hollow core linked to both office interior and the outside air ■ Make it even more efficient by increasing the internal surface which comes in contact with the air (the reverse radiator)
The need not to collapse in a fire	■ Use a beam with minimised heated perimeter over area ■ Apply fire resistant covering such as intumescent paint or fire-resistant board ■ Choose a beam made from concrete or use a vaulted masonry structure ■ For small projects, use timber beams and oversize to allow controlled charring
The need to avoid a 'hard' acoustic	■ Adopt a beam with a soffit profile that doesn't provide either acoustic focusing or flat panel reflections ■ Provide a beam/floor system with soffit coffers to accommodate acoustic absorption panels
The need not to block up our roads	■ Limit maximum beam size to about 18 m by 3.5 m to avoid the need for police escorts and oversize trucks
The need to reduce embodied energy	■ Choose beam material, sourcing, labour and fabrication complexity; transport methodology: maintenance materials and resources; re-use at end of working life; quantify and act on all of this at the time of deciding on structural system
The need to cut down on the use of zinc as a protection system	■ Do proper risk assessment of corrosion in marginal zones ■ Reduce total area of beam surface in at-risk areas to cut down protection
The need to reflect light around the space	■ Provide directed, sloping and curved soffit
The need to eliminate geometrical waste in all of the non-structural building components	■ Choose a rhythm which avoids cutting of materials by mapping to the minimum waste available modules of secondary components such as floor tiles, ceiling panels, façade glass, furniture
The need to use every part of the structure to help its neighbour	■ Look at load-sharing from beam to beam (needs width to provide torsional stiffness) ■ Look at using moment continuity over supports and into walls and columns
The need not to add material to prevent sagging	■ Get the rules changed: In floors, challenge the received wisdom about the limit for floor deflections (ask what does it really matter?) ■ To avoid over-designing edge beams, ask whether or not the façade really needs joints only 10 mm thick ■ Design a live-load sharing system between floors
The need for beauty	■ In the eye of the beholder, but has a fighting chance if natural rules and some of the relationships described here are allowed to express themselves

Table 2.5 The many complex factors which go into the design of a 'simple' beam

or a planet. All of them need to be designed somehow. But I know that at least if I can think my way through a beam design I might have some chance at doing something more complicated one day. I cannot run before I can walk. If cannot even design a beam, with all its contextual richness, what right do I have to be trusted with anything more?

I hope the thought process is clear. The need for a structure to resist the forces of gravity to hold up in the air a group of people sitting around work-desks begets an integrated conceptual response called 'beam' which only then can be tested using a combination of theory hard-won from engineering science coupled to equally important subjective value judgements. It is important to realise, as many do not, that you cannot just do the scientific tests, as tests have no purpose in practical real-world engineering without a conceptual proposition. A beam designed without context is as likely to fail as to succeed. This is why many wonder why they struggle to be 'conceptual' when they sit down in front of a powerful scientific testing tool like a computer. You cannot only test your way to a good answer no matter how big your hard drive is.

2.10 Enough is enough: A little challenge to use resources more wisely

Case studies into the material efficiency of typical buildings show results which highlight a bewildering iceberg of wastefulness in the construction industry. To take one revealing example: in a typical four-storey concrete frame building, less than 40% of the concrete in the columns is actually working. The received wisdom behind this is that it is 'cheaper to make all the columns the same size'. But the old timber shuttering system implied by this remark would be familiar to those Roman engineers who first used concrete in buildings 2000 years ago.

In ‘Enough is enough’, my Milne Medal address in 2010, I addressed the many factors which lie behind such engineering overdesign, from the institutional to the technical, from the constructional to the cultural. Far from being offended, many in the audience put their heads in their hands and said, ‘My Goodness, why are we behaving like this?’

The same case-study showed that 20% of the concrete in the floor slabs could be removed and they would still comply with the code. Much more interesting, and working from first principles in a way beyond the catch-all intentions of code, only 50% of the remaining concrete is actually useful – the rest is in a tension zone where it does nothing except add to the weight of the building and boost the load on the columns and foundations, so they also get bigger. So, all in all, only about 40% of the concrete in the building is actually working. The rest is just ballast. While this may be very good business for those who supply concrete it is not a wise use of resources.

While there is no single answer, it is clear that we have inherited a series of behaviours that lend themselves to high resource consumption. Some are cultural, like the familiar requirement for the structural engineer to build the design presented, rather than suggest ways of improving the efficiency of the design. But these behaviours are fairly and squarely in our hands for us to tackle.

So here’s a simple suggestion, a sort of ‘fight-the-flab’ for structural engineers. From now on you will only get Building Regulations approval if you *demonstrate that all of your structure is acting at say 90% of its capacity. And you should minimise the required capacity by using properly assessed loads, using contemporary materials technology and high quality construction techniques*. The onus should be on performance, on getting more for less, for the good engineer to prove that your structure sits in the ‘Goldilocks zone’ where it is just strong enough, but not too strong. Overdesign would become a professional offence. Overnight, this will change behaviour as engineers rediscover the satisfaction of getting a structure that is perfectly in balance with the natural forces acting upon it. Welcome to a new culture that will help flush out those apprehensive engineers who are not quite sure exactly what they are doing, so they simply put in more concrete, or more rebar, or more safety factors, to make sure that their ‘thing’ won’t fall down. This sustainable structural engineering should be a matter of professional obligation, and we could then hold our heads up to say we are taking responsibility for putting our engineering skill at the service of the needs of the planet, not just servicing the expectations of our immediate clients.

2.11 The importance of tools

Every generation invents tools, especially if there is an emerging market, or, sad but true, a war. The creation and effective use of tools is a key engineering skill, essential to the art of engineering. Each technological wave develops its own tools and does amazing stuff with them. Of course, one of the key late twentieth-century achievements of the structural profession has been the creation of digital tools that enable us to test, in a virtual world, structures of amazing complexity in a matter of seconds, and to go from drawing to fabrication and construction without ever picking up a pen. It is somewhat ironic that these tools free us from much of what we historically spent our time doing. So, the better our tools and the simpler they make our lives, the less secure are our (traditional) jobs.

On reflection, there are a couple of very useful tools for engineers that rise way above the rest and really stand the test of time: these are the pencil sketch, and the rule of thumb or the ‘back-of-an-envelope’ check. Every engineer needs to know what they are and how to use them.

2.12 The power of the engineering line

A good structural engineer needs to develop mastery of the engineering line. In project meetings you sometimes see engineers struggle to sketch something, and then give up as they mutter ‘I’m useless at drawing’. This self-deprecating remark masks a world of lost opportunity. In the hands of a proper engineer, a line represents everything, as powerful a communication device as has ever been invented. The engineer’s sketched line conveys a physicality as great as the table on which it is written, describes in two dimensions a space as three-dimensional as the building in which it is drawn, and carries natural forces as physical as the earth on which we stand. It is vital for engineers to develop mastery of the engineered line, for it is a code, shorthand for the physicality of things. To an engineer, a well-conceived line is our most powerful tool, far more powerful than any computer. We are lucky to be able to capture such complexity so elegantly, and I strongly believe that every engineer needs to train themselves to understand how to use such a line, so that eventually it is possible to improvise. At that point the world opens up and things that we once found tremendously hard become easy; mastery of technique flips it from being an impediment into a tool for invention and liberation.

As **Figure 2.6** illustrates, a sketched engineering line can be conceptually so strong that people, trucks and trains can sit right on top of it. In coded form, the line may rest upon mighty foundations, even though, of course, they are only sketched as delicate dots. Contained within a well-engineered line is the practical knowledge that the concept the sketch represents is actually able to be built. Furthermore it conveys the confidence that eventually, it will be possible to subject the concept to all necessary theoretical and cultural tests and eventually integrate the line, and the physical reality which grows out of it, into the whole project.

The medieval masters spent years agonising about this, and yet those engineers who today laugh off their sketching hamfistedness are guilty of dismissing it lightly. It is no coincidence that Leonardo spent many pages of his Codex notebooks

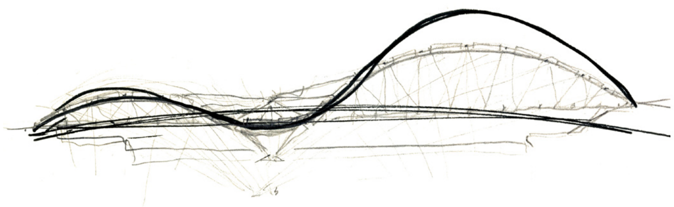

Figure 2.6 Engineering concept sketch drawn on a train by Chris Wise, leading to the final built project: Infinity Bridge, Stockton on Tees

using just this type of line: over and over again he explores with a single stroke of the pen why a ladder doesn't fall down when a man climbs up it, or how water flows down a plug-hole, or how light moves through the human eye. Historian Martin Kemp rightly describes these engineering drawings, in the right hands, as 'Theory Machines'. I am sure modern engineers are not intimidated by someone who died 500 years ago, so how interesting is it that, with the 'theory machine' still there right at our fingertips, we retreat so often away from drawing in favour of the robotic behaviour of the computer. So, more power to those engineers who have the confidence still to draw a line that speaks for itself, and who know the power and complexity of the instrument at their command.

Is the engineering line really that important? Well, some exceptionally gifted people, like Arup's late engineer Peter Rice, habitually linked technology and especially maths back into their minds to help them converge on an essential design truth that others could follow – his version of my engineering line I suppose. Often this cross-linkage gave Peter a very strong diagram, a robust core. He visualised the line as a trail, saying 'I'm a bit like a hound following a fox; I'm following something really close to the ground and I can't actually see where it's going.' This gave him a particular sort of dogged pursuit of the new, albeit one that required the luxury of a patron in the form of his host company.

At a more prosaic level, I am regarded as something of a free spirit, but nevertheless I am very effective on major engineering projects. This is in part because I can sketch. And I believe that the more complex the situation, of all the tools a structural engineer like me has at his disposal, it is this line that is the hardest to master, yet the most profound and useful. I should add that you don't actually have to draw the line – for other engineers like Peter Rice it is quite good enough to carry the clarity of the line in the mind's eye; it can be just as effective there.

2.13 The engineer's toolbox: Rules of thumb for testing

A structural engineer needs to use all the tools at his or her disposal to test a concept and turn it into a great piece of design. Some of these testing tools are formal pieces of theory; others are rough rules of thumb for quick conceptual design calculations. After a while it becomes clear that some techniques are used over and over again – in which case it is worth putting some in a conceptual toolbox and storing them on your computer and more usefully inside the head. These rules and aide mémoires are the basis of what used to be called 'fag-packet calcs' and are the second most useful real- time aid an engineer will ever have (after sketching).

Over the years, I have accumulated several pages of these things, but whenever I find something useful, I add it. **Table 2.6** shows the first few items in my list as a flavour. Once committed to memory they become like a musician's scales, fertile territory for improvisation, and very good for playing tunes with others.

2.14 Integrating construction with design

Who usually carries the biggest risk in a construction project? You only have to look at the efforts of Eiffage to get the Millau Viaduct constructed to realise the answer to that question. It is not the engineer with the huge insurance premium, or the architect. It is the contractor.

Steel and concrete limiting proportions:	**Simply supported beams**	**About 18:1**
	Continuous beams	About 22:1
	Cantilevers	About 7:1
	Trusses	About 30% deeper than beams
	Arches	About 6:1 to 8:1
	Catenaries	About 10:1
	Heavily loaded columns	About 12:1
	Lightly loaded columns	About 40:1
Watery rules of thumb:	Flow velocities in UK rivers	Typically between 0.5 and 3 m/s
	Nine out of 10 errors in flood estimation	Are a result of not measuring the catchment area correctly
	Flows in pressurised pipe systems	Normally have velocities between 0.5 and 1.5 m/s
	Pressures in normal distribution systems	Typically between 1 and 6 bar (10–60 m)
	Gravity drainage systems typically have velocities	Between 0.7 and 1.5 m/s
	Typical slopes in gravity drains	1:100 – 1:300
	Typical slopes on lowland UK rivers	Between 1:1000 and 1:300
Foundations and retaining walls		
Bearing capacity	Rock	2000+ kN/m^2
	Dense sand/gravel	200 to 500 kN/m^2 (10x SPT 'N' blows)
	Loose sand/gravel	50 to 100 kN/m^2
	Clays	25 (soft) to 400 kN/m^2 (very stiff)
Piles	Minimum spacing	3 diameters
	Bored piles in London Clay	(0.45 cu av x shaft perimeter)/3 + (9 cu base x base area)/3 where cu av is about 75 kN/m^2 and cu base is about 100 kN/m^2
	Bored pile diameters	From 300 to 1800 mm
Slope stability	Long term angle of repose:	Phi Φ, for sands and gravels use 25° to 30°
	Short term safe(ish) stability Sands and gravels: Clays: Rock:	1 vertical to 3 horizontal 1:1 Vertical but watch out for bedding planes
Pressures on retaining walls	Hydrostatic pressure	$= \rho wgh$
	Active pressure coefficient	ka=1-sinΦ 1+sinΦ
	Passive pressure coefficient	Kp=1+sinΦ 1-sinΦ
	At rest pressure coefficient	About 1.0
	Soil pressure (active, passive, at rest)	=k. ρsgh
Et cetera		
Et cetera		

Table 2.6 Sample rules of thumb for rough sizing

So naturally the structural engineer's knowledge of construction needs to be very high to bring down the overall risk. If only we could live up to that hope. At the end of the twentieth century, despite the best efforts of Michael Latham and John Egan in suggesting ways the design and contracting industries could work more closely together, and despite many corporate pronouncements that things have changed, lurking just underneath the surface is contractual confrontation, with claim and counter-claim when things get difficult. In other cultures, especially Japan, Germany, France and Spain, the contractor is involved early on, sometimes right from the start of the project, sometimes even as the co-promoter. Many of the best engineers in these countries work not for consultants, but for contractors. I think that that close relationship helps them become better, more useful, more imaginative, more capable structural engineers.

I make no apology for demonstrating this through the individual example of the great contributions of construction-literate engineering minds on our own projects: Julio Martinez Calzon in Spain when he helped the contractor Cubiertas y MZOV develop the final construction method to lift 3500 tonnes of floor structure in one go right up our 292 m Torre de Collserola Communications Tower. More recently Calzon worked with Bovis to realise our 90 m 'UFO' flying over the top of the redeveloped 100-year-old Barcelona Bullring; I think too of the persuasive work of Jorg Schlaich's office with Pfeiffer and ISG on the development of the cable net for our 2012 Olympic Velodrome structure; David Taylor of Dorman Long for his work on the erection methodology for the Infinity Bridge over the Tees; Bob Gordon of Bovis (now of MACE) for his thinking on the construction methodology which helped realise this pioneering technology for the Channel 4 cable-net wall in London. Their knowledge is a great and necessary complement to the theoretical, architectural and environmental thinking of a new generation of structural engineers.

The beauty of this mentality is that the engineer develops a design that is buildable. The down-side is that dealing in the high-risk world of contracting brings with it either high reward or high conservatism. And standing in the way, allegedly to preserve some sort of commercial competitiveness among the contractors, it is normal, at least in the UK, to delay the appointment of the contractor until the design is 'done'. This moment is somewhere near full working drawings, and the same system is very common in such places as Greece, Italy and the USA. In these countries, the contractor is only appointed after the design has been finished, which has two predictable consequences: either good ideas are brought to the project when it is too late, resulting in delay and redesign costs, or the design is developed with little understanding of new construction techniques and manufacturing technologies and so tends to perpetuate the status quo. In this traditional model, the unhappy consultant is regularly beaten up to produce an absolutely pristine set of working drawings and calculations, only for the first thing to happen once the contractor is appointed is a request for a redesign. While this is often based on a very good technological idea, or a practical appreciation of what is involved in building something, such an intervention by the traditionally appointed contractor comes too far along the project journey. As a side issue, given that all up fees on a building project are in the order of say 10%, even a small change so late in the day can demand significant extra human resources for a project, and although this is good for employment it is not good for the husbandry of engineering skill.

So, whenever I can, I argue as strongly as possible for the contractor to be appointed early. The neatest compromise is for this to happen with the contractor being paid for constructability and procurement input just like any other member of the design team. At that moment there is no guarantee that the contractor will get the project proper, so everyone is on their best behaviour for a while. Of course, as long as they are economic pragmatists who are judged on the financial success of each discrete project, contractors will tend to gravitate towards solutions that are familiar, or to manufacturing techniques they are comfortable with, or sometimes to those that are most profitable. But at least it is possible to bring that expertise into the team when the key design decisions are being developed and finalised, and that has to be much better than changing everything after tender. There is a side benefit of having a heavy-hitter like a major contractor in the team and that is that they provide a counter-balance to the wilder excesses of architectural imagination. This provides a dose of reality when it is most needed.

2.15 Emergent technology as an integrating force?

Emergent technology is the generic name for experiments that bring together the worlds of construction, engineering, architecture, evolutionary science, environment and computational software and hardware using the principles of genetic algorithms learned from Nature. It is inevitable that as such a new technological 'system' emerges, so does it test our art. I have been lucky enough to see this first hand as from 2003 I have been an external examiner on the Architectural Association's EmTech course run by Michael Weinstock, which is at the front of the field in world-leading experiment.

How might a whiff of 'emergent technology' add to the great body of human achievement? The very name hints at the creature from the Black Lagoon. The answer is being demonstrated by those very clever people who are currently exploring the world of genetic algorithms and their applications in the built environment. Some of these are nominally structural engineers; some architects; some environmental engineers. Yet they are doing something rather special, namely working across all of these disciplines as if there are no borders. I have seen these people develop a philosophical proposition that becomes an aesthetic one, which is then tested against environmental and structural principles before being squirted down the wire to a 3D printer which makes it large, ready for everyone to gawp at. And they do this as a small part of a two-year master's programme, all on their own. Then I go back to the multi-billion pound engineering scene which thinks that bubbledeck is the height of technical sophistication and really wonder if I'm on the same planet.

If I'm honest, I am less interested in the underlying maths than the possibilities it offers. I like to look for patterns, forms, an underlying holistic proposition, as Bronowski called it 'The Grain in the Stone'. And even if I don't quite understand all of the equations, I can gasp at the beauty of the answers. Probably this is a learned response, looking for something familiar, some cute elegance. I once saw the mathematician Marcus du Sautoy gradually add together lots of harmonics to get every single prime number from one to infinity. It had the air of a conjuring trick. He used a very clever algorithm that was beyond me. He managed to explain maths using music, prime numbers emerging out of musical notes, one of the unfathomable mysteries of the universe. To me, it seemed like magic. You just had to say 'Wow! How does nature do that?'

Who knows whether emergent technology will be the wellspring of our future projects? It will not be the whole answer but it is certainly able to open up new possibilities, less culturally conditioned. For some it certainly is already their drug of choice. But so far, emergent technology themes evolve painstakingly slowly, hindered by the limitations of the very systems of software and rapid prototyping that opened the door in the first place. We can be glad of the patience of these pioneers, and very optimistic for them, and us, as their tools get faster. Like any infant coming across something for the first time, many emergent projects are over-excitable and unsophisticated in the contextual understanding they bring into play. So although emergence offers great potential, it is culturally governed by the boundary conditions and success criteria we choose to impose, meaning that these need to be mastered if we are to get any intelligent answers from the technique.

Emergence has a serious pitfall when it comes to reality. It is this: the structural manufacturing sector, happy with its business model of millions of tonnes of steel and concrete, shows little interest in adopting principles of emergence whose end point is unclear and which might, perhaps, actually lead to a new sort of concept that reduces their sales of steel beams or pre-stressed planks. Emergence and its projects based on evolutionary design principles may eventually be very good for the planet, very good at reducing embodied energy, maybe better at producing more responsive buildings; but it will not flourish until the heavy-hitters think it will be good for 'business', and there we all have a challenge to produce the evidence and change the collective culture.

Today we see emergent phenomena without knowing whether they are in the mainstream or up a sideline. The next generation of these emergent tools should make us a wonderful alternative world of things that no one ever imagined could exist, but whether they pass all of the objective and subjective tests depends on the expression of need in the project brief. Choosing the rules we use for this, setting the criteria for acting as judge and jury: probably this is where we have the biggest development challenge.

2.16 Time for metamorphosis?

So, as structural engineers, what is the knowledge we really need? The professional institutions have stated their case and, as we have seen, they are not as definitive as you might expect. Useful knowledge for an engineer comes in many forms, from scientific fact to engineering theory to social awareness. To turn that into built projects in the real world, for the use and benefit of mankind, is down to each one of us to interpret. We each have to make a value judgement about what is important, and then act on that judgement. It is quite a responsibility.

As for the future, scientific theory does make predictions, and within its very narrow limits we know what the answer will be. But when it comes to complex life on this planet, and particularly human life going forward, there are bear-pits all over the place. Some have made predictions that, because of the status of the speaker, can be mistaken for knowledge. For example, in the late 1940s Thomas Watson Senior, then chairman of IBM, estimated that the world market for computers was about five (or was it six?). Others make statements about their own plans that, again, are taken as knowledge of what will happen because the speakers and the listeners believe they have the power and the authority to make it so. Perhaps 'Education, education, education' is a telling example that, like IBM's Mr Watson, good intentions aren't enough even for politicians apparently in charge of whole nations.

In response to the natural world, engineers have long cultivated a tradition of approximate knowledge. Even using the very best, most precise theoretical tools, their experience has learned through one disaster after another that we cannot know the exact answer and it is a jolly good idea to leave a margin for error. Engineers have learned not to trust the answers of their calculations absolutely, as they know they are only approximations to the complex interrelationship between the natural and human world, and that nature will always find them out if not treated with respect. How eloquent was the testimony to this in Japan in 2011 where the same structural engineers whose expert knowledge had helped them design a generation of buildings that could resist earthquakes and thereby save thousands of lives could only stand and watch powerless as thousands more were washed away by the ensuing tsunami.

So does our knowledge help us to know where structural engineers should be going? After 30 years as a reasonably successful engineering designer, the only thing I know for certain is that we cannot go back. But as for whether we need to become emergent technologists, technological gurus, global business people, or Specialist Generalists, I don't know. Probably we have the intellectual make-up to do all of those, or we could evolve it. But whether we have the awareness and whether we have the will – that is the question.

For the structural engineer this is the time for metamorphosis. As an 'Artist' I will just start. The 'Philosophers' among us will try to map out the path full of meaning. The 'Artisans' will wait for the path to be well-trodden, then follow it carefully, filling in all the gaps as they go along. As Einstein said, 'There comes a time when the mind takes a higher plane of knowledge but can never prove how it got there'. The structural engineer is dead…long live the structural engineer. I know that to be true but I cannot prove how I got there.

2.17 Conclusions

This chapter has shown that projects are complex, varied and require many different types of approach. It also explores some of the underlying patterns, then highlights the relationships within the projects that contribute to successful outcomes. Given the long time spans and apparent maturity of the structural engineering world, it could be assumed that the profession of structural engineering has reached a common position regarding the skills and techniques that are useful. However, this chapter has shown that the changing needs of society require considerable reflection and, most probably, the development of a new genre of engineers. To help in this development, this chapter attempted to tease out personal preferences towards working methods and includes some suggestions for the engineer's approach to the future.

If you answered mainly A you are a 'Philosopher';
If mainly B you are an 'Artist';
If C you are an 'Artisan'.
It is perfectly possible to be a mix, for example, an Artist with philosophical tendencies

Table 2.7 Results for engineering personality test

2.18 Note

[1] Please note the artwork for these chapters feature the author's hand drawings as would be done in practice during the design stage.

2.19 References

The Artist, the Artisan, and the Philosopher, plus the Engineering personality test was developed at the RSA's Royal Designers Summer School, co-directed by Ed McCann and Chris Wise, held at Minster Lovell, Oxfordshire in September 2007. Although never formally published, it is widely used by Ed McCann and Chris Wise in presentations and workshop sessions at their various teaching presentations. Ed McCann and Tim O'Brien developed the three personality themes at the Summer School. Chris Wise wrote the questions and drew the diagrams in this chapter, while Ed McCann produced the character summaries.

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Chapter 3

Managing risk in structural engineering

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CONTENTS

Any design is a process of risk management – organising resources so as to maximise the potential for success and minimise the potential for loss. Much of the time, the engineer is concerned with safety, durability and serviceability; for the user, this translates to reliable and cost-effective service in support of the building's function. Many day-to-day risks will be dealt with by any accepted and conventional design process, provided that process is suitable for the case in hand. Modern codes use more explicitly risk-based processes to provide standard numerical allowances to cover commonly experienced risks. More direct management of risk involves identifying performance requirements and the hazards that threaten them. By this means, the engineer and client can more closely appreciate how risks may affect the structure's use and how to control the risks to acceptable levels.

3.1 Introduction – the concept of risk

Risk is the combination of: the likelihood of an (unwelcome) event and the consequences of that event. Risk cannot be separated from any activity which has an eye to the future, be it the choice of lunch or the design of a bridge. In much commercial activity, risk is the other side of the coin from opportunity: every opportunity involves risk on the way to exploiting it and the risk involved needs to be kept small enough for the opportunity still to be worth it.

When talking about safety, the 'opportunity' is quite small as you cannot do better than no accidents. The possible loss is often substantial – at least in terms of the worst case consequences. But history tells us that thoughtful structural engineering and diligent construction keep the likelihood sensibly low.

Structures do not only aim for safety; support of the user's function, durability, landmark aesthetics and other criteria can all come into consideration. And in each case, the engineer needs to promote the potential for a better result and limit the risk of a worse result.

There are many summaries of risk classification and management approaches; for example, relatively high risk or sensitivity to risk may originate from:

- the structure being of unusually high value;
- the loading being unusually variable;
- the structural behaviour being unusually difficult to assure. Recognising when risk is rising to unusual levels is the first step in risk management.

Because every worthwhile activity involves risk, it follows that the sensible way of carrying out that activity is risk management. A design engineer using partial load factors is managing the risk of structural failure; putting a contingency into a cost estimate is managing the risk of cost overrun.

This chapter aims to raise the consciousness of risk management, not (necessarily) to change the designer's approach.

3.2 Risk criteria

In the UK, there is a universal requirement that risk shall be managed to the level which is 'as low as reasonably practicable' (ALARP). For detailed coverage, see HSE's *Reducing Risks, Protecting People* (R2P2) (HSE, 2001); broadly, it codifies the principle that a facility owner must protect the safety of anyone affected by the facility to the point where additional safety can only be obtained at greatly disproportionate cost.

It can normally be taken that design in accordance with authoritative codes, in the circumstances for which those codes are written, will achieve a result which is ALARP for the ordinary use of the structure. It does not guarantee adequate safety during construction or maintenance.

In addition, R2P2 offers a scale of tolerability of risk, on which, if the most seriously affected individual is subject to a risk of fatality of more than 1 in 10 000 per year, this is unacceptable. If the most seriously affected individual is subject to a risk of fatality of less than 1 in 1 million per year, the situation is broadly acceptable. This can be misinterpreted as requiring that a facility should control risk to a level below 1 in 1 million per year of any fatality, which is very much more onerous.

Box 3.1 Individual risk criteria

The figures given by HSE for the scale of Individual Risk, from 'broadly acceptable' to 'unacceptable' were arrived at by considering the risks to which people are exposed in normal circumstances.

Individual Risks (of fatality) averaged over the UK population, are approximately:

All accidents 1 in 4200 per year
Road traffic accidents 1 in 10 000 per year
Fall, etc. at home 1 in 8000 per year
Fire at home 1 in 70 000 per year
Accident at work 1 in 150 000 per year
Note: this varies greatly between occupations
Lightning strike 1 in 10 000 000 per year

On a recent project, the question was raised of what the appropriate risk criterion might be for the assessment of ship impact against the tower of a major suspension bridge over a navigable waterway. The basic individual safety criterion on its own would not provide an answer as it would be unlikely that any one individual would be so constantly exposed to the risk from ship impact that the risk to that person would be the determinant. Risk to occupants of vehicles on the bridge at the time of impact would be relevant and would inform the ALARP assessment – which would assess whether the cost to reduce the risk would be 'greatly disproportionate' to the value of the reduction.

The project, however, was big enough for the regional economic loss, in the event the bridge was unusable for a number of years, to be a major factor in the assessment of impact risk and prevention measures. It became a matter of the business case for the bridge itself and the risk appetite of the owner.

A more detailed discussion of structural failure risk, and the example of ship impact, is set out in Duckett (2004).

3.3 Acceptability of risk – appetite vs. aversion

In a commercial environment, it is conventional to treat risk in terms of willingness to take more risk with the prospect of more gain, or reluctance to take risk in favour of greater certainty of continuity. The shorthand term is 'risk appetite'. A company which favours lower risk and accepts lower profits is said to be 'risk averse'.

After the 2008 banking crisis, it was claimed that the banks were 'risk averse' when in fact they had been trapped by their 'risk appetite' exceeding their ability to quantify and manage the risks. Whether a company considers it has a high or a low risk appetite, ignorance of how its actual risk compares with its appetite is itself a risk.

In construction, the client is usually taking a substantial commercial risk in terms of developing a building for a future market. But most clients are risk averse in respect of structural engineering. They do not think that they are taking any risk in respect of structural safety or building serviceability.

Clearly there are exceptions; an authority commissioning major infrastructure in a seismic or flood prone area should be thinking explicitly about risk. It may be crucial that bridges and hospitals survive, in good working condition, events that cause extensive damage around them and, in such cases, the engineer needs to agree with the client whether the social need dictates an unusually low 'risk appetite' and therefore greater robustness in the structure.

3.4 CDM – construction, maintenance, refurbishment, demolition

The Construction Design & Management (CDM) Regulations (HSE, 2007) actively require the (UK) professional and management teams involved in construction to consider risk. The headline risks are to do with the construction process and the supply chain, but full treatment needs to consider maintenance, refurbishment (plant end-of-life replacements) and demolition.

The basic methodology of identification of scenarios and causes, evaluation of likelihoods and consequences and assessing the acceptability or otherwise of risks is set out later in this chapter.

However, the details of exercising risk management on the issues covered by CDM are covered in specialist publications at greater length than can be included in this manual. Further reading is given at the end of this chapter.

3.5 Construction time cost and buildability

The expression that 'time is money' and the triangle of 'time, cost, quality' are commonplace. The completion date and budget are major risk elements to any client, be they domestic, developer or public authority. Failure to open and start generating revenue can have massive effect on the client's finances and credibility – as Eurotunnel found when the opening of revenue services through the Channel Tunnel was delayed by a year and the extra debt almost crippled the company. The design team should therefore be aware of the priority that the client is putting on time and budget constraints.

Risk may arise in site operations or far back in the supply chain. Clearly adverse weather at the site can reduce the rate of construction – and the client needs to know whether the form of contract includes that risk or transfers it to the contractor. For a long-running project, fluctuations in the world price of cement, steel or indeed labour can all affect the outturn cost and, again, the client needs to know where the risk is being carried.

While the likelihood of each potential adverse event may be estimated with a 'spot value', such as the probability of occurrence during the contract, the 'potential loss' item in the risk assessment has two measures – duration and cost. Risk will need to be evaluated and quoted in both ways, so care is needed to keep the risk assessment even-handed.

The management of construction stage risks might include workshops during the design to examine the procurement of materials and equipment, erection plant and methods and so on. An experienced developer or project manager will participate in such workshops. A less experienced client might not want to sit in on the workshop itself but must be made aware of what the conclusions are and what measures are being taken to control the time and cost risks.

In a commercial context, 'risk treatment' describes the options and strategy employed by the client, abbreviated as Avoid, Control, Accept or Transfer (ACAT):

> *Avoid* means modifying the project in order that a potential scenario does not arise; an example might be to abandon the use of an imported product so as to avoid supply chain, transport or import controls.
> *Control* might mean modifying the design so as to ensure 'open market' availability of plant, for example, reducing component lifting weights to allow a wide range of cranes.
> *Accept* might mean the client recognising that uncertainties in contract outturn cost will be prohibitively expensive to lay on the contractor and therefore are best accepted as part of the client's own contingencies.
> *Transfer* is the opposite of Accept – to pay for the contractor to carry certain risks in order to lower the contingency element of the project budget.

Both the design and the contract conditions will be affected by the client's strategy for risk treatment.

3.6 Service loading, statics and dynamics

In principle, the essential balance between a structure's minimum strength and maximum load is made on a probabilistic basis and therefore there is a risk that – using all the appropriate factors – the load will exceed the strength. Structural engineers spend a great deal of time calculating loads and strengths with factors applied to give a reasonably cautious balance and without being concerned with the risk of the balance tipping the wrong way.

Part of the reason is that the codes are not always explicit on the risk which is implied. For example, EN 1991-1-4 (Wind effects; BSI, 2001) gives a reference value of wind speed 'with an annual risk of being exceeded of 0.02'. It is implied that the lower the annual risk (that the owner is prepared to accept), the higher the equivalent threshold value which should be applied in the design. Conversely, the higher the value of load you design for, the less likely it is to be exceeded.

While the wind code gives a value with a 'risk' – more correctly, a probability – of 0.02 of being exceeded in any year, it does not limit the amount by which the value may be exceeded. Codes mostly do not give the relationship between the higher threshold value and lower annual probability, although research papers may do so.

Loading of other kinds may also vary enough to exceed normal allowance:

- Soil pressures vary with the natural variability of the material.
- Groundwater pressure varies with depth of the zero pressure surface.
- Building floor and roof loading varies with management of use and of additions, etc.

In each case, the 'normal' design basis will be established either by reference to an authoritative code or by local accepted practice. In either case, it is open to the engineer to change the risk by changing the design value. As an example, in the 1980s the water table under London, which had been lowered by abstraction over many years, was found to be rising as a consequence of reduced industrial consumption. As a result, the British Library, with an unusually long expected life, needed additional precautions in basement design to deal with both the known conditions at the time of construction and possible additional demands if the water table continued to rise unchecked.

Seismic loading is a particular area where risk is obvious; in the UK, most building designers would assume that the demands of any sensible seismic loading (given the expected life and use of a building) would be met by a conventional design against wind loading. For a nuclear power plant, however, the sensitivity of the use means that the exposure to seismic load needs to be addressed. As with wind, the higher the load that is designed for, the lower the annual probability that it will be exceeded. The dynamic character of seismic effects, however, also mean that simply adding structural strength may change the loading. A thorough risk treatment of seismic exposure and performance involves considering both the magnitude and the frequency spectrum of ground motion, to confirm that the structure adequately covers the potential challenge.

Other dynamic loads may present a range of structural and performance problems: oscillating wind loads on lightweight towers and chimneys are probably the most common. But even footfall on lightweight floors, crowd movement on grandstands and dancers in clubs can deliver 'excess' deflection, vibration, stress and fatigue beyond what an equivalent (assumed static) load would indicate.

Box 3.2 Uncertainty and variability

Uncertainty can be reduced by investigation – the more knowledge the engineer has on the uncertain issue, the fewer unknowns remain and therefore the lower the risk. Variability will remain although, in the structures we specify, better quality control will reduce the variability.

What the number and range are of vehicles that might use a bridge is uncertain, but this can be resolved by investigation of traffic. What speed the vehicles will travel at is uncertain, but may be possible to moderate by design features.

3.7 Structural capacity and ductility

Structural capacity should be more predictable than loading; the size, shape and materials of the structure are in our hands to specify. However, there are limits to what we know of structural capacity.

Firstly, there are unknowns in the capacity of heritage structures and the many existing structures which it is the engineer's task to evaluate, modify and/or strengthen for re-use. Secondly, there are unknowns in the capacity of foundations and earthworks where natural materials are the important structural elements. Thirdly, even concrete and steel have ranges of actual strength compared with the strength assumed in normal calculation.

In general, a new-build structure of conventional form and materials will have a capacity assured by practice, research and quality control. The body of knowledge built up through research and through experience of satisfactory use in the past underwrites the allowances in codes by which the 'minimum' strength implied by specification is further reduced by a factor to allow for exceptionally poor materials and for poor quality in construction.

Another factor which protects the engineer and owner from low capacity structure is that most of the common materials have a reserve of ductility through which 'failure' means a degree of distress as the excess load is shed to less loaded elements or the peaks of dynamic load disappear. Steel in tension or bending, reinforced concrete in bending, timber and even masonry exhibit this behaviour to some degree.

As a result, even when a structure is poorly built or loaded beyond its design intent, the result is not necessarily collapse but may be repairable damage. However, some materials and modes of action do not show ductility, and it is crucial not to use the assumption of ductility in the wrong circumstances. Examples might be:

- buckling behaviour;
- brittleness and fatigue vulnerability in welds;
- monolithic glass.

A combination of redundant structural form, more conservative load factors and quality specification will be needed to counter the higher risk of inadequate capacity.

Different structural forms will also behave differently 'beyond design basis'. While the limit stress might be the same in a cantilever, a simply-supported beam and a redundant frame, the redundant frame will withstand very much more mistreatment than the others.

3.8 Robustness and extreme challenge

The topic of robustness is covered in detail in a later chapter in this manual (see Chapter 12: *Structural Robustness*).

Extreme challenges are those rare loadings or excitations under which one might not expect a structure to come through unscathed, but which a robust structure will survive with damage that can be repaired. Conventionally, the 'design basis' is a series of loadings which, at the normal serviceability level, can be supported by the structure without damage. Robustness comes into play when the loading is 'beyond design basis'. In the previous section, ductility and redundancy were identified as properties with universal value in limiting the risk from 'beyond-design-basis' events.

In cases where the use of the structure makes damage limitation an important criterion, the engineer should be prepared to assess performance against a probability-based range of extreme challenges. For example, a hospital in a seismic zone could sensibly be designed for minimal damage (and survival of all services in working order) in an event that would do major damage to ordinary buildings.

A similar process applies to sensitive buildings in cyclone areas: the structure and fabric need to withstand high wind, heavy rain and impact from wind-driven debris in a way that other buildings may not.

In order to manage the risk to the community, it is necessary to set more stringent criteria of risk to the structure.

3.9 Codes of practice

In the UK, codes of practice adopted an implicit risk management approach to structural design with the use of ultimate load criteria and partial load factors from the 1970s onwards.

Partial load factors recognised the separate contributions of load variability and capacity variability. In particular, they encouraged the recognition of situations where one load countered the effect of another, in which the margin of difference would vary more dramatically than where one load added to the effect of another.

The UK and EU have now adopted the Eurocode series, for which the risk background is discussed in guides and papers such as Calgaro and Gulvanessian (2001) and Gulvanessian and Holický (2005). The Eurocodes have not only introduced a comprehensive structure of partial load factors but also classifications of building structures in terms of Consequence Class and Execution Class.

Consequence Classes CC1 to CC3 describe buildings of increasing risk sensitivity, basically by the number of occupants of a single structure. This starts the design process on the basis that increased sensitivity to risk should be reflected in lower probability of failure in any element and under any loading pattern. It is open to the engineer to extend this thinking, especially to non-building structures.

Execution Classes EXC1 to EXC4 reflect not only the Consequence Class of the building, but broad categorisation of the load conditions (static or dynamic) and form of construction (such as welding) and the resulting Class carries with it implications of construction quality appropriate to the management of the risk of failure.

Eurocodes provide an organisation of the concepts contributing to the definition of loads, the assessment of performance and the criteria of adequacy. For an engineer using Eurocodes, any risk management work which contributes to the structural design should, for preference, be translated into the parameters used in the Eurocodes rather than developed as a separate analysis.

For engineers working under other code regimes, the same logic applies but will necessarily depend on the extent to which risk concepts have been worked into the code requirements.

Box 3.3 Hidden risk factors: tolerances, workmanship

In general, deviations in dimension and quality – within the bounds allowed by normal specifications – are taken as covered by load factors. However, the adverse effect of tolerance on structural performance is rarely if ever covered explicitly.

For example (from a recent investigation) the quoted tolerances on levelling the subgrade below a ground-bearing slab and on levelling the top of the slab resulted in the slab itself not having the intended thickness and being inadequate for the intended loading. A deliberate policy by the contractor of reducing the slab concrete in favour of cheaper fill was suspected as part of the problem, but the combination of tolerances ('permitted deviations') had ultimately produced an inadequate structure.

It is valuable to check, during the design, that the structural form, details and performance are not critically dependent on close control of dimension or quality. When such control is needed, the design information and specification are the means to manage the risk, although it may also be appropriate to make the design more robust to reduce the sensitivity to error.

3.10 Innovation

Innovation can take the engineer into the realm of 'unknown unknowns'. History shows that in some cases, the adoption of logically justified forms or details gave rise to issues of understanding which had not previously been significant – shear buckling in box girders, for example. On the other hand, the widespread workmanship problems in precast concrete panel systems around the same time might have been envisaged with imagination at design stage.

By their nature, Codes of Practice cannot be expected completely to cover innovative structural forms. For that reason, the engineers need to deploy both understanding of principles and explicit risk management methods, such as those described in the following sections.

3.10.1 Methodology – hazard identification and logging

For any engineer carrying out explicit risk management, there is no substitute for maintaining a Hazard Log (or Risk Register, the terms are effectively interchangeable). A Hazard Log summarises:

- The causal factor generating risk
- The scenario in which failure and/or loss occurs
- The measures in place which restrict causation or loss
- The type and size of loss
- The measures of likelihood, loss and – in combination – risk
- Intended action if the risk is to be reduced

Hazard identification (Hazid) is the process by which the Hazard Log is first populated. It may be a simple process of running through a check list based on experience, which can be done by one or two people. In more complex or sensitive structures, it is sensible to mobilise the experience and imagination of several people in a workshop; the check list becomes an agenda for the workshop.

Either way, the Hazard Log is less important than the process of questioning the normal assumptions to determine whether there are unusual risk issues or to confirm that conventional design and analysis will be enough. (This is written in terms of structural design but the process holds good for the CDM risk register or a project time and cost risk exercise.)

A graphic matrix of likelihood and consequences is commonly used to put a coded risk marker in the hazard log. In **Figure 3.1**, 5 is the high end of the likelihood or consequence scale, 1 is the low end. So 5 in likelihood might represent 'several occasions in the life of the structure' and 5 in consequence might represent 'collapse or 100% loss of use for several years'.

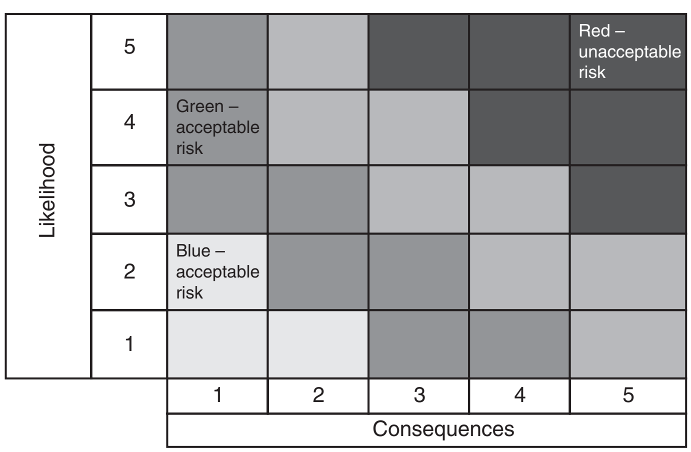

Figure 3.1 Example Risk Classification Matrix

Hazard Number	Hazard	Hazard Situation Description	Cause of Hazard Being Realised	Consequence of Accident	Control Measures	Inherent Likelihood	Inherent Severity	Risk Ranking	Risk Control Action
	Collapse	Retaining wall under permanent horizontal pressure from soil & groundwater	Unexpected geotechnical/ hydrogeotechnical conditions	Collapse of retaining wall; multiple fatalities	Conventional design in accordance with civil engineering standards including for robustness in face of element failure. Ground conditions are predictable and well documented. Design includes acceptance of major rise in ground water level	1	5	T	Conventional requirement for structural asset monitoring
	Corrosion	Retaining wall under permanent horizontal pressure from soil & groundwater	Wall not watertight. Leakage of water through retaining wall. Water on inner face affects fixings, system boxes	Asset damage. Possible failure of fixings, system boxes fall from wall onto maintainer	Conventional double skin construction with drained cavity protects systems. Water collected and pumped out.	1	2	N	Maintenance inspections
	Wet surface	Retaining wall under permanent horizontal pressure from soil & groundwater	Wall defect and high water pressure, e.g. neighbouring water main burst	Water standing on/ flowing over floors/ platforms; accumulating on tracks Slips & falls	Conventional retaining wall design & double skin construction with drained cavity; floor drains and track drains to pumped sumps.	1	3	N	

Figure 3.2 Example Hazard Log. This example is for the retaining walls to an underground station. The measure of severity is in fatal casualties; asset damage is not estimated

The codes represent degrees of concern: dark grey (usually in red) would be unacceptable risk, but medium grey (usually in green) and very light grey (usually in blue) would be acceptable risk. In the example shown in **Figure 3.2**, a 3-point scale has been adopted (Intolerable, Tolerable and Negligible).

Some users like to combine the numbers for Likelihood and Consequences by addition or multiplication, and then use the result to put the risk on a scale where one value of result always represents one severity of risk. Since the numbers themselves represent ranges of value, this can produce inconsistent results; the greatest consistency is found when each number is the same factor up from the previous one (e.g. factors of 10) and the numbers are added. In the matrix in **Figure 3.1**, 5+1 gives the same result as 3+3.

Such matrices are easy to misuse. No one likelihood or consequence scale is appropriate to all projects. Having defined scales, the colour codes in a matrix like the one in **Figure 3.1** may be correct for one project but not for another – the acceptability of combinations of Likelihood and Consequence need to tally with the client's risk appetite, for example. In the case of a hospital in a seismic zone, quoted above, the unacceptable area would cover more of the matrix than would be used for an office.

If using Hazard Logs and risk codes, calibrate the evaluation and then use it consistently.

3.10.2 Methodology – mathematical methods

In the coding methodology outlined above, one line across the matrix represents a range of likelihoods: 'several occasions in the life of the structure' translates into an upper bound of perhaps 1 per 10 years (0.1 per year) and a lower bound of perhaps 1 per 75 years (0.013 per year).

If a more rigorous mathematical treatment is needed, reference should be made to textbooks on statistical and probabilistic methods such as Melchers (1999) or one of the Eurocode commentaries listed at the end of this chapter.

One powerful approach is to use a Monte Carlo add-in to a conventional spreadsheet. In this analysis an input parameter which has some variability is replaced by a statistical distribution (e.g. a normal distribution for concrete strength). The add-in software repeatedly recalculates the spreadsheet using different values for the statistical inputs, controlled by the defined distribution. At the conclusion, the chosen outputs are described by mean values and graphical/numerical distributions from which can be derived (e.g. the value which has a 5% probability of being exceeded).

The advantage of this method is that it can be used with many statistically varying inputs without concern for mathematically combining statistical functions. It is used successfully on cost estimates to avoid adding quite arbitrary contingency figures.

The disadvantage of this method is that, by specifying many variable inputs, the analyst can conceal the fact that some variables will change together in the same way. For example, if the cost of construction labour rises, practically every item in a cost estimate will increase. But if the analyst does not model this, the mathematics will largely offset the increase and give a misleadingly 'accurate' result.

3.10.3 Methodology – risk management in the design process

The methods set out in the previous sections enable the design team to identify, describe, classify and – if need be – quantify risk in terms of safety, cost, programme or asset damage. Those processes encourage the designer to consider whether risks are excessive or, even if not excessive, are worth working on to reduce.

Fundamentally, this is a matter of setting up design options and analysing them – as designers conventionally do – but using risk criteria alongside cost, durability, performance and the like.

There are several ways to subdivide risk management measures: one such is Eliminate, Reduce, Isolate, Control (ERIC):

- To eliminate a risk is to form the structure so that the hazard generating the risk is no longer relevant; for example, to lay out a vehicle manoeuvring area so that no important columns can be struck.
- To reduce a risk is to take special measures, such as increasing the strength or adding barriers so that, in the same example, the impact may happen but is unlikely to result in significant damage.
- To isolate a risk is to limit the damage to an affordable enclave, so significant damage may be done to the structure but the building use can continue while the structure is repaired.
- To control the risk is to accept that damage may occur but to minimise the effect on occupants.

When explicit risk management is a part of the design process, it becomes a feedback loop in which, if a risk is identified as sensible to reduce, the engineer makes the changes (or develops the options) and the risk assessment is repeated to enable adoption of the best risk-reducing design. The hazard log becomes the record of the process.

3.11 Risk management – conclusions

Unusual levels of risk can arise through sensitivity of the building's use, variability of load and uncertainty in structural behaviour.

Many instances of risk higher than normal can be managed by analysis of the unusual conditions and appropriate use of load factors, as envisaged by risk-informed Codes of Practice. However, codes provide a grounding of accepted practice; the risk management is in the work of the engineers thinking about the design and the particular features of the structure.

Where it is necessary to work outside the pattern of code design and analysis, explicit assessment of risk issues, event likelihoods and consequences will be appropriate. The scales and combinations of likelihood and consequence need to be confirmed for each project, to make sure that acceptability matches the client's risk appetite.

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[1]*Ed Tufton sadly passed away in June 2012. ICE Publishing would like to thank Ed, for his valuable contribution to the manual, and his colleague, Charles Milloy, who kindly proofed this chapter on Ed's behalf.*

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Chapter 4
Sustainability

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Sustainability is a phrase that is both familiar and open to interpretation. This chapter provides an overview of what sustainability means for a structural engineer. It sets the scene by considering an overview of global and home policy, discusses how sustainability can be measured and then addresses the basic principles of sustainability for buildings throughout the design and construction process. Carbon reduction policy is briefly reviewed, including the Brundtland Report to the Climate Change Act and Part L Building Regulations. The key measurement tools are introduced including footprinting, BREEAM, LEED and CEEQUAL. The main focus of the chapter is the application of the policies and tools for low carbon design of buildings for the structural engineer. Implementing the integrated design approach is discussed using case studies throughout the design process and each section concludes with significant 'sustainable wins'.

CONTENTS

4.1 Introduction: putting sustainability into a global context

Sustainability is a legislative moving target; global carbon reduction policy is in a constant state of flux. However, whilst the numbers of targets may change, the principles of sustainable practice are core to achieving any target and do not change. Sustainability and climate change have moved to the political forefront of global development since the 1970s. Following on from the Green Movement and the oil crisis (1973) the World Commission on Environmental Development (WCED) commissioned a report on sustainable development entitled *Our Common Future*, the so-called Brundtland Report (1987).

Since then, various political agreements and organisations have come into fruition, the most notable being the Kyoto Protocol (Kyoto Agreement, 1997), the Sustainable Development Commission (SDC) and the Intergovernmental Panel on Climate Change (IPCC). Today, we speak of a low carbon economy, greener lifestyles, climate change adaptation and mitigation, but what does this mean for the construction industry? In order to answer this question we need to consider what are the tangible components of sustainable development.

4.2 Sustainable development and policy

The overall aim of sustainable development is 'to enable all people throughout the world to satisfy their basic needs and enjoy a better quality of life, without compromising the quality of life of future generations' (Defra, 2005, p. 6).

At a national level the UK Sustainable Development Commission's legislative response to the Brundtland Report has been to identify and clarify five guiding principles (as shown in **Figure 4.1**). These five principles are intended to lead the UK Government, Scottish Executive, Welsh Assembly Government, and the Northern Ireland administration towards a broad and overarching understanding of the term sustainability. For example, sustainability is much more than just quantifying carbon reduction; it also requires analysis of current statistics on world population, natural resources and the threat of climate change.

The five principles provided the background for the UK's *Energy White Paper: Empowering Change?* (HM Government, 2003). That document outlined a pathway to reducing carbon emissions to comply with the Kyoto Protocol targets. The UK targets were set at achieving 60% carbon reduction from the 1990 levels by 2050 and recently this target has been increased to 80% reduction by 2050. To achieve these targets new legislation has followed (such as the Climate Change Act 2008, the Sustainability Act 2003, the Housing Green Paper and the Energy White Paper 2007). The percentages of carbon reduction required by the Building Regulations are shifting targets. Part L was revised in 2010 to ensure a 25% reduction whereas a 44% reduction is expected in the 2013 regulations. There are also discussions about Part A Building Regulations being revised to take into account carbon dioxide emissions created through the embodied energy of materials and during construction.

The built environment is being targeted because the construction and operation of buildings account for approximately 50% of UK carbon emissions. Commercial buildings are responsible for approximately 30% of carbon emissions. The government wants all new non-domestic buildings to be carbon zero by 2019 but new-builds can neither compensate for, nor replace, existing building stock. The carbon adaptation factor of these buildings will be subject to structural advice and refurbishments. The commercial viability has been outlined by the Investment Property Forum (IPF, 2009) and, on the domestic scale, the UK Housing Green Paper *Homes for the Future* (DCLG, 2007) has pledged to build two million new homes by 2016 and three million by 2020. All new homes are to be zero carbon from 2016. The existing domestic stock has been targeted through Local Authorities and energy supply companies. The construction industry therefore has a responsibility to

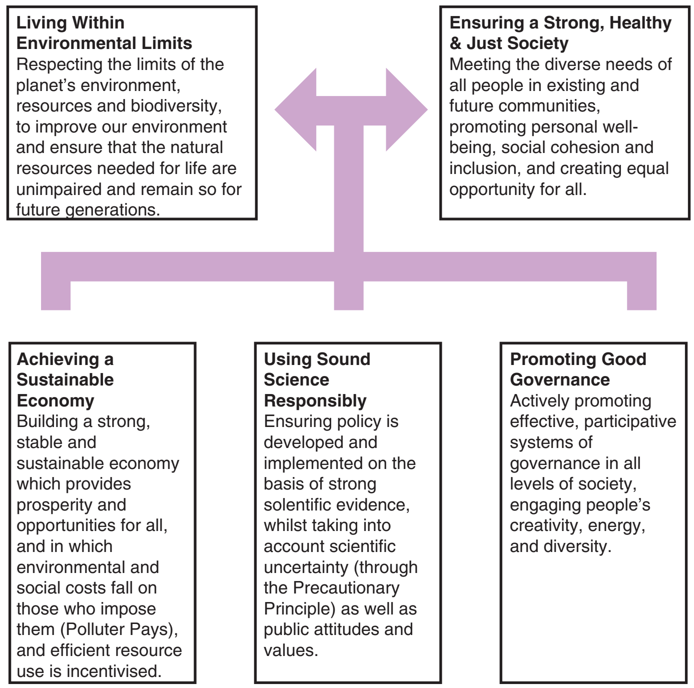

Figure 4.1 Shared UK Sustainable development Principles (Defra, 2005 © Crown copyright 2005)

reduce carbon emissions for both new and existing commercial buildings and dwellings.

Low carbon buildings conjure up various images from mud huts to intelligent buildings that sense daylight, people and overheating; however neither is possible without a client who is willing to invest in the level of specification that is required. To create the required differentiation to invest in sustainability, known values of measurement can be used. It is attractive to a client to invest in a building with a good life cycle cost, with a return on their investment and which can be labeled with a BREEAM Rating.

Following the introduction of the European Performance Building Directive (EPBD) we can label our buildings and appliances with an A* to F rating, with A* being energy efficient such as a Code For Sustainable Homes Level 4 and an F rating being inefficient and representative of our existing building stock. In the UK, only 1% of the building stock will have been built to these new standards, while the remaining building stock is predominantly E to F rated. The new Code level 4 and 6 homes and BREEAM Outstanding buildings are not commonplace. Energy efficiency and retrofit is the new important market.

New-build commercial developments that have been successful at achieving carbon targets are exemplified by the Adnams Distribution Centre, G. Park Blue Planet, Vulcan House – UK Borders Agency, Manchester Civil Justice Centre and the Mapungubwe National Park Interpretive Centre. On the domestic scale, homes can achieve zero carbon which has been seen with the zero energy development housing scheme at Beddington known as 'BEDZED' and the Hockerton Housing Project in Nottinghamshire.

For low carbon buildings to become mainstream we must acknowledge that sustainability is a global commodity; there is a market demand for sustainable solutions but clients are deterred from making an investment when there is no qualitative differentiation. What creates the qualitative differentiation is when the targeted measurement system is used at the beginning and measured accurately to demonstrate a good financial investment. In design terms, a typical naturally ventilated office costs £6/m^2/yr compared to an air conditioned office at £20/m^2/yr (ENCON 19) but the naturally ventilated office can only be achieved if designed in that way. We need to build and develop contracts with these targets in mind to create sustainable solutions today for tomorrow. The following section introduces a variety of measurement frameworks on which to hang sustainability, and identifies key sustainability design factors for everyday design use.

4.3 Is sustainability measurable?

To function well in the modern marketplace an engineer needs to be familiar with the wide variety of methods used to measure and assess sustainability to deliver sustainable solutions. This section starts with the key definitions of ecological footprints, embodied carbon, operational energy and Key Performance Indicators, and progresses to sustainability measurement frameworks including BREEAM, LEED, Green Star and Code for Sustainable Homes.

4.3.1 Ecological footprint

A term defined by William Rees and Mathis Wackernagel, an ecological footprint is measured in hectares to calculate the human demand on the biosphere. The calculation includes food, water and energy to ultimately indicate what land area is needed to support a population. Ecological footprinting science is an important tool because it underpins the mechanism for carbon trading and management on global, urban planning and human scales.

Designing with an ecological footprint target will determine how a project can reduce its impact; starting from the design concept of increasing site-yield by giving every square metre a multiple use and continuing with how waste, energy, resources and water can be reduced throughout design, construction and operation of a project's life. For more information see ISO 14001:2004 (BSI, 2004) and the websites concerned with ecological footprints in the references.

4.3.2 Carbon footprint

A carbon footprint is a measurement of how we produce and use energy with reference to the impact on global warming. It is measured in kilograms of carbon dioxide (emitted due to our energy activities) released into the atmosphere per year.

In a building context, engineers must be aware that carbon dioxide (CO_2) can be emitted into the atmosphere through a variety of mechanisms other than by simply burning fossil fuels to provide a power supply to a building. In simple terms the carbon emitted would include the burning fossil fuels in transporting construction workers and manufacturing materials both on-site and off-site. Once all the contributing factors of carbon have been identified, the total carbon footprint can be calculated. This leads quite naturally into discussing the terms 'embodied' and 'operational carbon' which are covered in the following paragraphs. For more information see ISO 14064-1 (BSI, 2006c), ISO 14064-2 (BSI, 2006d), ISO 14064-3 (BSI, 2012), ISO 14065 (BSI, 2007), PAS 2050 (BSI, 2011), PAS 2060 (BSI, 2010), CEN/TC 350 – Sustainability in Construction Works.

4.3.3 Embodied carbon

The definition of embodied carbon is broken down into direct and indirect embodied carbon. Direct embodied carbon relates to the energy involved in how the construction components are transported to site and the operation of putting the components together. Indirect embodied carbon relates to the energy put 'into' the component itself in terms of extracting from the ground, processing and manufacturing materials. It also includes any energy used to transport subcomponents or equipment in any of these stages.

Embodied carbon has a significant impact on a building's carbon emissions and this proportion has been steadily increasing over recent decades as technology has developed. In addition one might also consider the recurring embodied carbon which is defined as the energy required for maintenance, refurbishment and replacement of components during the lifetime of the building. The ratio of embodied carbon to operational energy has grown to approximately 40:60 (as shown in **Figure 4.2**); a level that requires attention similar to that of the Part L Building Regulations and will be addressed in revisions of Part A Building Regulations.

Embodied carbon is controlled through the choice of construction materials and structural engineers are crucial players in reducing direct, indirect and recurring embodied carbon. Reduction can be achieved through adding in the 'carbon factor' to how the structure is framed, the simplification of components and the use of local materials. **Figure 4.3** shows the typical embodied carbon impact of the building elements.

A further step may be taken by considering how the structural system can be refurbished, i.e. what components will need replacing first and whether they can be removed without too much deconstruction or use of energy. Finally, the material

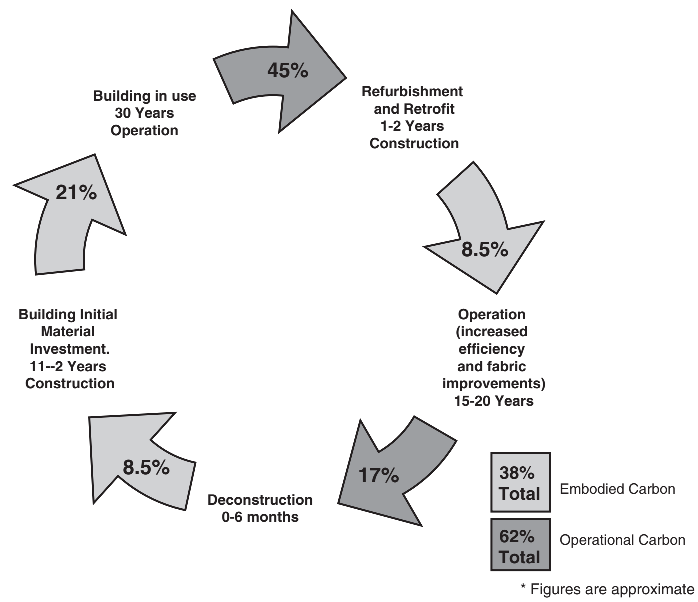

Figure 4.2 The carbon life cycle of a typical building

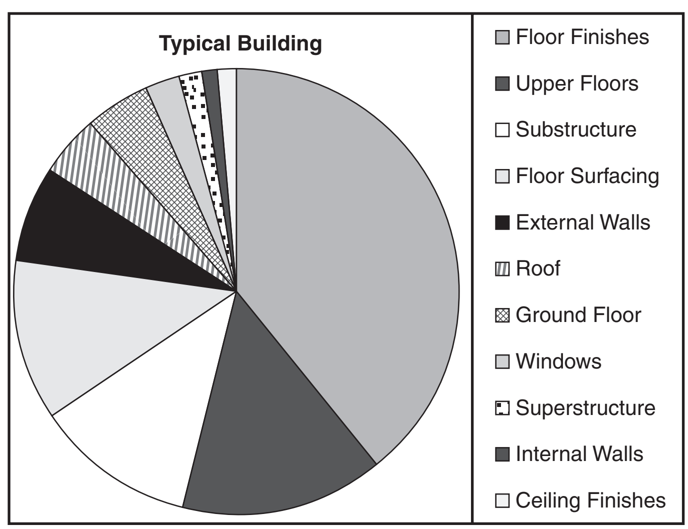

Figure 4.3 Contribution of building elements to typical building impact (Anderson *et al.*, 2002; reproduced courtesy of Wiley-Blackwell)

choice itself partly determines the level of embodied carbon: the typical design materials of steel, concrete, glass and masonry are high in indirect embodied carbon content whereas at the lower end of the spectrum will be materials such as timber, recycled products or alterative materials such as adobe bricks and straw. These are the choices which the structural engineer will need to make. Therefore it is our role to consider the specification of products, responsible resourcing and work with materials that where possible negate carbon and consider how we can reduce energy use in construction to complement health and safety requirements.

4.3.4 Operational carbon

In a building the dominant source of carbon emission is from how people use the building, the energy supply and the energy associated with the running the building's systems throughout its lifetime. Reduction in operational carbon can be achieved through adopting passive design techniques, energy efficient systems and cleaner fuel but often it is not commercial to reduce electrical, heating and cooling loads for the building user (unless they are the client).

Methods to reduce operational loads can be futile without the involvement of the building occupant as early as possible. Understanding how the building will be used through user engagement will define the environmental conditions for comfort as well as providing information on how they will use the building in the long term. Equally, users will develop an understanding of the proposed building systems and have the opportunity to appraise designs to create something that really works for both client and user, with cost benefits. Creating flexible and adaptable environments will create a space that can become reused, but the ultimate test is putting in the appropriate technology for future use.

Achieving a reduction in operational energy leads to structural engineers, architects and building service engineers merging at the building skin to discuss airtightness, thermal heat transfer and robust detailing. Building façades are a structural envelope as well as dictating how people use space and energy inside the building. For a low carbon building the façade can optimise the use of daylight, natural ventilation and solar gains to reduce electrical, heating and cooling loads. In some cases the façade dictates the building form. We can see this through architectural history where in the Georgian era large floor-to-ceiling window heights were the norm to increase daylight, and then to reduce heat loss at night shutters and thick curtains were used.

Where structural engineers think about operational energy some fascinating buildings can be produced – as seen with the Institut du Monde Arabe in Paris by Jean Nouvel and the Centre Pompidou by Renzo Piano, Richard and Sue Rogers, Norman Foster, Edmund Happold and Peter Rice. However, a more recent building where the building form was defined by its constraints and client-requested low running costs is the ING bank in Delft by Alberts & Van Huut Architects. The building has been built next to a busy road where acoustics are an issue, but this went against the clients' desire to have a naturally ventilated and daylit building. The solution was for the building skin to behave as an acoustic barrier, which was achieved through the external wall being constructed of masonry bricks to absorb the sound and then built at a slope to reflect the sound waves. This allowed the designers to significantly reduce the operational energy of the building by maintaining a natural ventilation and daylight solution.

Structural engineers will find it useful to consider using the following measurement tools: Simplified Building Energy Model (SBEM); Standard Assessment Procedure (SAP); Dynamic Simulation Software; and Energy Performance Certificates (EPCs), all of which are discussed further below.

4.3.5 Energy performance ratings in buildings

The European Energy Building Directive (EPBD) came to fruition to drive forward energy efficiency and has led to methods of calculating energy use in buildings and revisions in the Part L Building Regulations 2002. Energy performance described by the Directive Implementation Advisory Group (DIAG) is required for the following building types:

1. Display Energy Certification (DEC) to be displayed in buildings larger than 250 m² that are occupied by a public authority.
2. Energy Performance Certificates (EPCs) to be displayed in commercial buildings larger than 250 m² that (a) are frequently visited by the public and (b) where an EPC has previously been produced on the sale, rent or construction of that building.

EPCs rate buildings subject to heating and electricity consumption, but this performance can be related back to building form and construction. Electrical consumption focuses on lighting, small power appliances, and how the electricity has been generated including renewable technology. On a domestic scale the NHER have developed a Standard Assessment Procedure (SAP) for dwellings which is also incorporated into the Code for Sustainable Homes. For non-domestic buildings, the Simplified Building Energy Model (SBEM) process is used. To achieve a good rating takes building science to a dynamic level where the interaction between the construction and building systems can be modelled and the emissions ratings calculated.

As regulations tighten their carbon reduction requirements the initial strategy of achieving carbon reduction through improving the efficiency of building systems and energy-saving technologies will not be enough. Radical steps will have to be taken with construction and building form which will require design teams to work more collaboratively to achieve requirements. In construction, developments have been made with Modern Methods of Construction which have been developed through research at Nottingham University and the BRE Innovation Park.

Introducing the measurement of energy performance as a design parameter early on in the design process will lead to a good efficiency rating and define what construction, building form and modern methods of construction are suitable solutions.

4.3.6 Key Performance Indicators (KPI)

Since the late 1990s, the construction industry has been benchmarking environmental performance of projects with Key Performance Indicators. Any project over the value of £100k is measured using KPIs which are monitored through the measurement of the reduction of waste removed from site, energy and water used during the construction process as shown in the 2009 *Construction Statistics Annual* (Office for National Statistics, 2009, Table 16.6) and reproduced in **Table 4.1**. This has led to the introduction of Site Waste Management Plans, Environmental Impact Assessments and the Considerate Contractors Scheme.

More recently in 2005 the UK Government's Sustainable Development Strategy launched 'Securing the Future' (Defra, 2005) which asks the construction industry for developments that deliver sustainable, towards-carbon-zero and zero-waste solutions. To achieve these targets we must adopt them at the beginning of the design process so they become a part of the project itself and build in cost savings.

4.3.7 Measurement frameworks

In the previous section, we discussed some clear-cut ways to measure sustainability. However, in recent years a plethora of assessment methods have begun to dominate the market in different regions of the world. The assessment methods all have a similar structure, that in essence establishes a set of appropriate categories and assigns points to each category. Each category has a weighting which is a subjective measure of how important that category is deemed to be. The weightings are used to produce a final value (usually a percentage) which represents how well a design or building matches the criteria. The most used and widely accepted methods around the world are BREEAM (UK), CEEQUAL (UK), LEED (US) involved and Green Star (Australia) and **Table 4.2** outlines the categories involved and provides example weightings used in each method.

4.3.8 Building Research Establishment Environmental Assessment Methods (BREEAM)

BREEAM is the commonly used assessment method for buildings in the UK and rates buildings from Outstanding to a Pass. In the 1990s, the Building Research Establishment (BRE) established an Environmental Assessment Method which was to be used voluntarily on projects to measure sustainability in the built environment. It initially focused on offices, schools

		Performance				
KPI	Measure	2005	2006	2007	2008	2009
Impact on the environment						
– product	% scoring 8/10 or better	53%	54%	51%	55%	64%
– construction process	% scoring 8/10 or better	44%	45%	44%	48%	49%
Energy use (designed) - product	Median energy use kg CO_2/100m² gross floor area	4291	3729	3775	4474	4539
Energy use – construction process	Median energy use kg CO_2/£100k project value	293	293	273	192	241
Mains water use (designed) – product[2]	Median water use m³/100m² gross floor area	53.2	52.0	90.4	80.0	49.5
Mains water use – construction process	Median water use m³/£100k project value	8.2	8.9	8.2	7.1	6.3
Waste – construction process	Median waste removed from site m³/£100k project value	41.6	37.0	39.1	36.9	36.6
Commercial vehicle movements – construction process	Median movements onto site/£100k project value	29.4	30.4	29.4	26.5	28.3
Impact on biodiversity						
– product	% scoring 8/10 or better	33%	36%	34%	35%	31%
– construction process	% scoring 8/10 or better	45%	48%	46%	49%	47%
Area of habitat – created/retained – product	% reporting no change or an increase in area of habitat	76%	83%	78%	80%	82%
Whole life performance – product	% scoring 8/10 or better	41%	41%	39%	35%	44%
1	KPI data for earlier years can be found in previous editions of Construction Statistics Annual.					
2	Limited data use with caution.					

Note: Sample sizes and distribution of data between construction sectors for some Environment KPIs have not yet stabilised.

Table 4.1 Summary of industry performance from 2005 to 2009 – Environment KPIs (*Construction Statistics Annual*, 2009) © Crown copyright 2009

BREEAM (UK)	Arbitrary units	CEEQUAL (UK)	%	Green Star (Aus)	Arbitrary units	LEED (US)	
Management	12	Project management	10.9	Management	10	Sustainable Sites	Dynamic, context dependent weighting system
Health and Wellbeing	15	Land Use	7.9	Indoor Environment Quality	20	Water Efficiency	
Energy	19	Landscape	7.4	Energy	25	Energy and Atmosphere	
Transport	8	Ecology and biodiversity	8.8	Transport	10	Materials and Resources	
Water	6	The historic environment	6.7	Water	12*	Indoor Environmental Quality	
Materials	12.5	Water resources and water environment	8.5	Materials	10	Locations and Linkages	
Waste	7.5	Energy and Carbon	9.5	Land Use and Ecology	8*	Awareness and Education	
Land Use and Ecology	10	Material Use	9.4	Emissions	5*	Regional Priority	
Pollution	10	Waste management	8.4	Innovation		Innovation in design	
Innovation	10	Transport	8.1				
		Effects on Neighbours	7.0				
		Relations with local community and stakeholders	7.4				

Table 4.2 **Collated international assessment methods sustainability measurement categories**
*** Regional weighting**

and domestic buildings. However, it has since developed BREEAMs for courts, healthcare, industrial, multi-residential, prisons, retail, communities and international projects. More recently in the UK, the Code for Sustainable Homes and the Code for Non-domestic Buildings have been developed with the Department of Communities and Local Government.

A BREEAM assessment is a two-stage process, with an initial design and construction review and then a post-construction review. Credits are awarded subject to demonstration and evidence that they have been complied with through drawings, calculations, specifications, client requirements and contracts. BREEAM Manuals provide designers with guidance on best practice and technical references but also provide a baseline for sustainable buildings.

It is becoming more common to find Local Authorities asking for BREEAM as a planning requirement; already any project in Wales has to achieve BREEAM Excellent and the requirement is no longer voluntary. (For more information see www.breeam.org.uk.)

4.3.9 Civil Engineering Environmental Quality Assessment Award (CEEQUAL)

CEEQUAL was launched in 2003 with the support of the ICE, CIRIA, CECA and ACE to assess environmental and sustainability performance in civil engineering and public realm projects. The assessment method has identified the indices shown in **Table 4.1** to measure environmental performance in civil engineering projects. CEEQUAL relies on clients, designers and contractors to go beyond the legal and minimum requirements and consider environmental parameters throughout the design, specification and construction process. Projects suitable for a CEEQUAL assessment are predominantly non-building related projects. (For more information see www.ceequal.co.uk.)

4.3.10 Leadership in Energy and Environmental Design (LEED)

LEED was established in the United States through the United States Green Building Council (USGBC) in 1998. The assessment process is about the measurement of sustainability and considers the whole life cycle of a project to determine a sustainability value, and is therefore able to account for the client's investment in economic and ecological terms. LEED certification adds economic value through focusing the ecological aspects of creating a healthy living and working environment for the building's occupants therefore increasing their productivity and satisfaction. This approach provides a framework to identify and implement practical and measurable solutions to allow the client to understand the yield from the investment in a sustainable building and encourages client participation. (For more information see www.usgbc.org.)

4.3.11 Green Star

Green Star has been set up by the Australian Green Building Council to address the carbon emissions from the built environment. Green Star was launched in 2002 to tackle commercial buildings which account for 8.8% of Australia's national greenhouse emissions. The assessment method has developed from BREEAM and LEED and follows the same principles shown in **Table 4.1**. (For more information see www.gbcaus.org.au.)

4.3.12 Code for Sustainable Homes (CSH)

The building of domestic properties constitutes a major portion of UK construction sector activity, and as such was seen as one of the main targets for improvement.

The BRE's EcoHomes guide (Rao *et al*., 2003) was established in 1990 to provide guidance on how to build homes in a sustainable way. The guide was superseded by the CSH for new domestic properties in October 2007.

The CSH (DCLG, 2008) is a design guide produced with the aim of helping UK housing developers to achieve zero carbon emission levels by 2016 (HM Government, 2008, 2009). In terms of energy performance, a 'Level 1' home corresponds to basic UK Building Regulations Part L compliance, 'Level 4' is a Passivhaus standard and 'Level 6' represents a zero carbon development.

Currently, use of the CSH is compulsory in the design of social housing, which must achieve a minimum of code Level 3 or 4 by 2010, Level 5 by 2013 and Level 6/zero carbon by 2016. The CSH has stipulated that the energy and water credits are mandatory, and the code level can only be achieved if the energy and water credits comply with the corresponding performance targets.

To promote the winning of credits engineers should consider *The Green Guide for Specification* (Anderson *et al*., 2002) which rates the proposed construction thermally and ecologically at the structural design concept stage.

The CSH approach is being adopted in the Code for Non-domestic Buildings which will become future practice.

4.4 Implementing an integrated design approach

4.4.1 What is a sustainable building?

A sustainable building is something that goes beyond the targets and asks how the building responds to its site, geographically, climatically, socially and economically: questions that, when answered, will lead to the beginnings of a sustainable building. A sustainable lifestyle is a further step that will lead to self-sufficiency and energy autonomy. It is perhaps useful to describe here what a truly sustainable building would be like.

Sustainable houses have been built by and large from materials sourced locally and/or materials whose thermal behaviour is appropriate to the site and climate. They may even include materials that absorb CO_2. Materials which have low-embodied carbon (i.e. those requiring only a small amount of energy to process, transport and manufacture them) are used where possible and are chosen primarily for their thermal performance as required by the local climate and site. The building is designed to be flexible enough so that daytime solar heating is maximised and night-time heat losses are minimised. An example of this technique would be to use Trombe walls which, due to the material's specific heat capacity, absorb and effectively store heat releasing it slowly back into the building during evening hours. **Figure 4.4** shows how the Trombe Wall theory can be adapted for roofs.

Water consumption can be reduced through the use of a greywater system and solid waste can be fed back into the nitrogen cycle. The whole design of the building is focused around avoiding the need for significant heating, lighting, ventilation and water requirements. The building is also adaptable and flexible for all seasons using design features implicit within the building envelope. As the thermal envelope of buildings become more airtight and thermally efficient, the internal air quality will have to be maintained through increased and adaptable natural ventilation and possibly through the use of hygroscopic materials which absorb excess humidity and improve air quality.

4.4.1.1 Case Study: Hockerton Housing, Nottinghamshire

Passively designed buildings such as the Hockerton Housing Project have been designed to use solar energy to heat the building. The project to build five homes was completed in October 1998, with each home costing £90 000 to construct (costs were driven down by it being a self-build project). One of the passive techniques adopted included the use of south-facing sunspaces to harness the heating capacity of solar energy. The heat is absorbed into exposed masonry walls and released when the

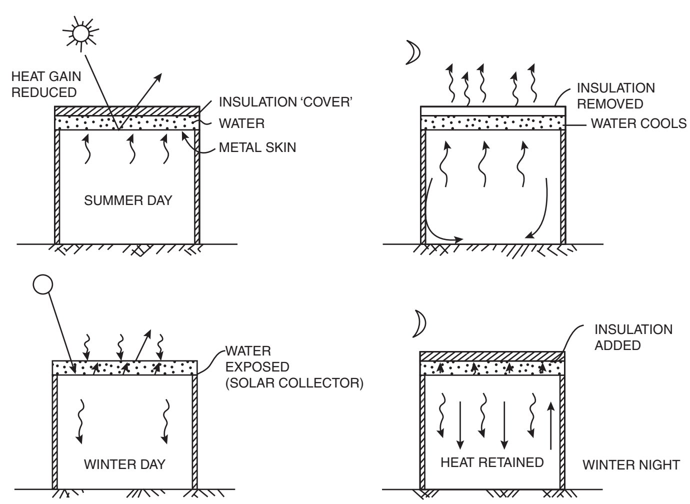

Figure 4.4 Trombe wall theory adapted for use on a roof

internal ambient temperature reduces, causing the wall to act as a radiator. Energy consumption is further reduced by the houses being partially buried and through 'super' insulation. Electrical energy has been reduced by optimising daylight to reduce the lighting requirements and the use of energy efficient appliances. Hockerton Housing also achieves energy autonomy with photovoltaic cells and on-site wind turbines. Further to their energy autonomy, residents also provide almost all their food themselves, compost their food and waste, and use a reed bed system for their liquid waste.

With this description and case study in mind it is clear that sustainability is a very broad term, encompassing many issues. Founders of Constructing Excellence, John Egan and Michael Latham, argue that early collaboration between designers leads to a more successful outcome in terms of reducing cost, improving health and safety and achieving client satisfaction. Despite commercial pressures, achieving sustainable building is the ultimate outcome. If sustainability is not given due consideration from the outset of a project, the likelihood of delays being incurred and abortive work increases.

The structural engineer who understands how the web of decision-making affects the embodied carbon, operational energy and environmental impact of a building can bring immeasurable added value to the conceptual design process. The basic philosophy the structural engineer needs to adopt should consider the whole life cycle of a building because at each stage of its life there are associated issues involving energy usage and delivering Brundtland's vision of sustainable development. Facilitating the shift to sustainable development comes through informing contracts and procurement routes that build sustainable principles into the design process.

To demonstrate the best value, long-term costs must be calculated to inform the decision-making process throughout the project's lifecycle. The benefits of a long-term approach allows maintenance and operational costs to be further considered and discussed at project inception stage, potentially influencing the design solution. Adopting this approach will demonstrate the value of integrating renewable technology, water recycling systems, responsible resourcing and passive design.

Typically used as a financial measurement of asset management, the whole life cost refers to the total cost of an asset from inception to decommissioning. For a construction project this would mean the cost associated with planning through to replacement as listed below:

- planning;
- design;
- construction/acquisition;
- operations – occupation and how the building is used;
- maintenance – comfortable operation, refurbishments and life cycle costs;
- renewal/rehabilitation;
- financial (e.g. depreciation and cost of finance); and
- replacement or disposal (see Chapter 5: *Taking a through-life perspective in design* for more detailed coverage).

At an early conceptual design stage structural engineers can readily make a beneficial impact on site selection and many others areas but most notably in four categories, which are categories within all the assessment methods described in **Table 4.1**:

1. energy and CO_2 emissions
2. materials
3. surface water run-off
4. waste

We will discuss each of these categories in the context of the design process and describe how a structural engineer can have a significant impact on the credits awarded by these methods during the design stages.

4.4.2 Conceptual design

Sustainable buildings encompass many aspects which need to be considered at concept stage to define the design philosophy and identify measurable targets. To provide an overview of design factors, **Table 4.3** is a design matrix of sustainability to facilitate how the multidimensional goal of sustainability may be quantified. We can identify that some issues are site-wide and others more technical. As the design process progresses, the focus will shift to technical issues.

4.4.2.1 Design factors

Site selection typically occurs before the structural engineer has been appointed; however, early investigations into the site's geology and climate can inform the design brief and define design parameters. Early investigations can provide insight into what renewable technology is appropriate for the site, geotechnical suitability for foundation solutions, and what material can be sourced from the site. It is also recommended that the regular utility service checks, flood risk assessments, ecological surveys and archaeological surveys are carried out too.

4.4.2.2 Focus 1: Energy and carbon dioxide (CO_2) emissions

Energy and carbon dioxide performance is weighted heavily in all the assessment methods due to the direct relationship between operational energy and carbon emissions. In principle, the more efficient the thermal envelope and airtight the construction, the more the operational energy demand is reduced. Furthermore, operational energy is reduced by adopting passive design techniques that optimise natural daylight, thermal mass, natural ventilation and passive construction. Operational energy consumption of the building is also directly related to its context (the site location and the local climate) for example, an exposed site on top of a hill near the coast will be windy

Negative	Sustainability Spectrum	Positive
	Materials	
Imported materials		Indigenous materials
High embodied carbon materials		Low embodied carbon materials
Non-renewable energy materials		Renewable materials
Non-recyclable materials		Recyclable materials
Toxic materials		Non-toxic materials
	Land Use	
Destroys rich soil		Protect/creates rich soil
Destroys nutrients		Creates/adds nutrients
Produces no food		Produces its own food
Destroys wildlife habitat		Provides wildlife habitat
Uses high productivity land		Uses low-productivity land
	Urban Context	
Favours high energy transport		Favours low energy transport
Favours polluting transport		Favours non-polluting transport
Excludes urban agriculture		Includes urban agriculture
No open space		Forever preserved open spaces
Destroys human habitat		Provides human habitat
No solar and wind access		Zoned for solar and wind access
	Water	
Destroys pure water		Creates pure water
Wastes rainwater		Stores and uses rainwater
Ignores greywater use		Uses greywater use
Waste run-offs		Creates percolation
Obtains water far away		Obtains water locally
	Waste	
Dumps black water		Recycles black water
Wastes embodied carbon		Recycles embodied carbon
Dumps solid waste		Recycles solid waste
	Air	
Destroys clean air		Creates clean air
Pollutes air thermally		Avoids thermal pollution
Pollutes indoor air		Purifies indoor air
	Energy	
Wastes solar energy		Uses solar energy
Ignores buildings' thermal inertia		Uses buildings' thermal inertia
Dumps waste energy		Recycles waste energy
Wastes wind energy		Uses wind energy
Wastes biomass		Uses biomass
Ignores daylighting		Uses daylighting
Ignores natural ventilation		Uses natural ventilation
Intensifies microclimate		Moderates microclimate

Table 4.3 Sustainability matrix (McDonough, 1992). Reproduced courtesy of William McDonough & Partners

Negative	Sustainability Spectrum	Positive
	Responsibility	
Destroys silence		Creates silence
No participatory design		Participatory design
Needs frequent repair		Maintains itself
Addictive and enslaving		Enlightening and liberating
No response to nature		Responsive to nature
No response to change		Responsive to change
No response to culture		Responsive to culture

Table 4.3 (cont.)

and therefore have increased heat losses from the building fabric due to wind behaviour. In such circumstances it would be beneficial to reduce the impact of the heat losses by protecting the building from wind. Building earth bunds or vegetation barriers could contribute to the overall design solution.

Carbon emissions in commercial and domestic buildings are calculated using different methodologies but each includes an assessment of fundamental building – physics principles. In commercial buildings the calculation method used is the Simplified Building Energy Model (SBEM), which measures carbon emissions by calculating the percentage improvement of the Building Emission Rate (BER) over the Target Emissions Rate (TER). In domestic properties the calculation method is known as the Standard Assessment Procedure (SAP) where the Dwelling Emissions Rate (DER) substitutes the BER and the percentage of improvement is the DER over the TER.

There are alternative, more rigorous methods, which involve further building physics; for example, the use of Dynamic Simulation Software can lead to a better assessment of the overall energy performance and carbon emissions.

The construction solutions of tomorrow will be heavily influenced by building science and bioclimatic design and will need the structural engineers to provide input on materials, not only their structural properties but also their thermal behaviour. How the individual elements fit together will need to be in accordance with certified robust methods detailing how to reduce sound transfer and heat losses.

There are many research projects in the UK and Europe such as the Passivhaus project that explore passive design techniques. The project originated in Germany and was established in the UK in 2001 with the project Cost Efficient Passive Houses as European Standards (CEPHEUS). To achieve the reduction in carbon emissions to Levels 5 and 6 of CSH standards, Part L (2010) and 2013 Building Regulations, the construction type will need to move away from the traditional masonry cavity wall to modern methods of construction such as Structural Insulating Panels (SIPs), Insulated Concrete Form (ICF), insulated solid wall construction, Phase Change Materials (PCMs), or thermal mass techniques such as Trombe walls.

During the conceptual design stage the operational energy of the building needs to be considered and actions taken to reduce it. This requires a significant amount of modelling using quick tools such as SBEM, SAP and the LT Method. A number of examples showing how this can be done to increase the credits gained in this category are provided below:

- Designing the building to ensure that there is sufficient natural ventilation using stack effects. If, right from the conceptual design stage, the appropriate ventilation is ‘designed in’ then less of an active energy requirement will be needed to ventilate the final design. A good example is the natural ventilation approach applied to the Inland Revenue Building, Nottingham.
- Considering the modern methods of construction combined with producing a façade that is adaptable by taking into account its response to orientation, the local climate and seasonal variation. In order to make these decisions, the early conceptual designs will need to be modelled to assess their thermal performance. It may be that some insulation will need to be removable, and innovative materials might be used to ensure sufficient natural sources of heating and cooling.
- Ensuring the rooms are sized correctly with sufficient glazing to ensure natural daylighting. Once again the design will need to be modelled at the conceptual design stage in order to ensure this.

Early decisions made in consultation with a sustainable building structural engineer will produce a building with a low operational energy. If a building is designed passively so that it requires less operational energy then local renewable sources of energy become much more cost-effective and the building may then have the potential to become effectively carbon-neutral.

4.4.2.3 Focus 2: Water and surface water run-off

All construction projects have an ecological impact, the effects of which can more often than not be detrimental to the water and nitrogen cycles through land-use depletion. Sustainably focused projects will look at how this impact can be reduced. It is also important to minimise surface water run-off at source as this can help reduce flood risk downstream; such measures reduce the amount of water discharged into surface water drainage systems, thus helping to minimise infrastructure costs. For sites in the UK areas designated by the Environment Agency as high flood risk, credits can only be awarded if 100% of surface water flows are discharged through an attenuated system and even for sites where low risk attenuation is sort. Refer to the CIRIA Interim Codes of Practice for Sustainable Drainage and Planning Policy Statement 25 – Development and Flood Risk (PPS 25) for further guidance. Evidence is required for the assessment methods to include calculations for retention and attenuation systems, flood risk assessments, drawings and correspondence confirming that appropriate authorities have been consulted and the proposals approved.

Methods could be as simple as installing a Green Roof, minimising building contact with the ground or opting for a permeable surface car park. Reducing the ecological footprint of the construction site will lead to a reduction in surface water and localised flash flooding. Such measures should be given due consideration at the earliest opportunity as the associated storage and treatment plant to facilitate the above may have some impact on the building layout and structure.

From the site-wide issues to building systems, the reduction of water consumption in buildings is also targeted by the assessment methods and using CSH as a guideline, domestic water consumption is targeted at 80 litres/person/day. By reducing water consumption, not only do we use less but we also reduce the impact on the existing discharge rates into the sewers.

4.4.2.4 Significant sustainable wins

1. Early site investigations which are carried out in accordance with BS 5930 Code of Practice for Site Investigations (BSI, 1999) and where appropriate the parties have been consulted to identify if the site is ‘of historical interest’, in an area of Outstanding Natural Beauty, of archaeological interest or particular architectural character.
2. Reduction in water consumption is achieved through the specification of water-saving devices, e.g. tap specification and low flush toilets. Further reduction can be achieved through the introduction of rainwater harvesting and/or greywater recycling although such options have greater financial implications.
3. Surface water run-off is reduced by the integration of Sustainable Urban Drainage Systems (SUDS), soft landscaping and reduction of hard surfaces, Green Roofs and attenuation systems.
4. Optimising the building layout for best use of daylight and outdoor views; materials that promote good indoor air quality; materials that can comply with acoustic requirements (Building Regulations Part E).

4.4.2.5 Case Study: Wessex Water

A successful building project where energy targets and whole life performance have been included in the design brief is the Wessex Water Headquarters Building (Bennetts Associates Architects, Buro Happold Engineers) which was completed in 2000. Ten years later this project is being used by the Commission for Architecture in the Built Environment (CABE) as a case study for sustainable design (see www.cabe.org.uk/case-studies/wessex-water) as it is a project where every aspect of carbon emissions was considered from concept to demolition.

For example, during construction, embodied carbon was reduced by using a bus designed to carry bikes up hill so that the site operatives could go to site on the bus then cycle home, downhill. However, what makes this project iconic is the approach taken to the building design; a good site analysis was carried out at an early design stage, which led to the building using the site’s topography and climate to its advantage. Consequently the architectural form was orientated to the south, and was narrow in width to make best use of natural daylight and to allow natural ventilation. Natural ventilation was further enhanced by using a floor-to-ceiling height of 3.2 m with an exposed concrete soffit, and excessive heat gains were reduced by installing solar shading. All these steps have contributed to reducing the building’s operational energy to 51 kWh/m^2/yr and carbon emissions to best practice levels for naturally ventilated offices (Jones, 2008).

4.4.3 Detailed design

4.4.3.1 Design factors

Existing buildings represent 80% of the building stock in the UK. Although the focus is currently on new-build legislation, existing buildings also require investment to reduce their carbon footprint. Hypothetically speaking, raw material availability is limited and many metals are becoming scarce resources, such as indium, zinc, hafnium and terbium (Cohen, 2007); according to the *Construction Statistics Annual* (Office for National Statistics, 2009), the UK imported £12 billion of raw materials and exported £6 billion. We need to be mindful that the cost of materials will continue to rise in line with the cost of transportation and energy costs in manufacturing processes. This cost could be reduced through using materials that are locally resourced, from waste, reclaimed or on-site. To use these types of material we have to consider that the choice can dictate the structural framing solution yet structural rationalisation can still be achieved.

The assessment methods are quite limiting and restrict the awarding of credits to the reuse of building materials, retention of building façades and the refurbishment of existing building structures. The starting point is to think ‘how can this building be reused, how can the impacts be reduced, are the building loads fit for purpose?’ and the engineer should not feel constricted to using these ideas just in order to gain credits. Sometimes the assessment methods do not cover good alternative sustainable options.

Designing with materials is a large component of the structural engineer’s repertoire but for sustainable buildings this choice must include responsible sourcing, opting for low embodied carbon materials, and adopting a ‘Reduce, Reuse, Recycle’ philosophy to lead to minimal-zero waste solutions.

These basic principles are outlined in the following section of this chapter.

4.4.3.2 Focus 3: Material choice

Engineers are aware of the technical parameters of optimising the use of a site and the framing solutions but they are often not familiar with how the materials are used with the mindset of reducing their impact on the embodied carbon of the construction process.

Reducing embodied carbon from the outset involves choosing a method of construction that is led by the materials' response to the local climate, resource availability, recyclability, toxicity and renewability. In simple terms, low embodied carbon construction would comprise of local sources and waste products typically represented by strawbale, rammed earth and limecrete and hempcrete construction (which extract carbon from the atmosphere). Next in the spectrum of embodied carbon we have recycled materials such as reclaimed bricks, recycled steel, reused timber, recycled aggregates, glass and then any product manufactured without the use of fossil fuels. At the opposite side of the spectrum we see steel, clay-fired bricks, concrete and petrochemical-based materials.

Often engineers wait for the architect to specify the materials but we equally dictate the materials required structurally. As custodian of sustainable material use, it is important also to use the BRE *Green Guide to Specification* (Anderson *et al.*, 2002) where construction types are rated from A* to E on their environmental impacts. There are also other guides available from the Green Building Store, Eco-Merchants and NGS Green Spec; in addition, manufacturers are producing more ecologically responsible materials for walls, roofs, windows and floors.

Responsible resourcing may start with the *Green Guide* but it should also be commonplace in our specifications, from source through to the supply chain, and to how the kits of components fit together on-site. For example, the embodied carbon in concrete can be reduced through the use of ground granulated blast furnace slag (GGBS), lime, pulverised fuel ash (PFA), and secondary and recycled aggregates which often result in increased compressive strength and reduction in curing time. The BS 8500 series for concrete specification (BSI, 2006a, 2006b) allows alternative concrete mixtures to be considered outside of Environmentally Controlled Construction (EC^2) and BS 8800. Responsible sourcing of timber is covered by using Forest Stewardship Council (FSC) certified timber or endorsed timber under the Programme for the Endorsement of Forest Certification (PEFC) which ensures that the timber used is managed sustainably and does not use endangered species such as mahogany or other tropical rainforest hard woods. Other materials such as steel, glass and masonry can be responsibly managed through using products that are manufactured by EMAS and ISO accredited companies.

Embodied carbon is not the only property that should be considered; at any early stage it is recommended that construction schemes are optioned not only with structural efficiency but also operational energy in mind. A low carbon building will consider the material's colour (emmissivity), thermal conductivity (U-value) and thermal lag (decrement factor) because they affect the thermal response, and thus operational energy, of a building. For example, only 50–90 mm of material is useful in thermal mass; more than that could be detrimental but without early modelling these decisions cannot be assessed.

From using materials to reduce carbon through to responsible specifications, reducing material consumption can be achieved using waste products. To facilitate the recycling and reuse process, if you are not already doing so, treat any construction project as a kit of components that can be deconstructed with minimal waste.

4.4.3.3 Significant sustainable wins

1. Material specification where 80% of the building complies with *Green Guide* Rating A.
2. Consideration of the constituent materials and their volume.
3. Responsible sourcing – BES 6001 – Framework Standard for Responsible Resourcing of Construction Products (BRE Global, 2009).
4. Reuse of at least 50% of the existing building façades and reuse of 80% (by volume) of the existing building structure.
5. Designing for robustness and easy refurbishment.
6. Understanding the insulating properties of construction and natural materials.

4.4.3.4 Case study: WISE at Centre of Alternative Technology

The Centre of Alternative Technology (CAT) has built the 'Welsh Institute of Sustainable Education' to showcase sustainable design and materials that are not in mainstream construction as shown in **Figure 4.5**. 'Treading the earth lightly' was achieved through using lightweight materials so that strip footings could be used. Superstructure was FSC certified timber frame with solid timber floors. The use of cement in the concrete was minimised by replacing it with hydraulic lime or GGBS and secondary aggregates. Wall construction was a combined use of rammed earth construction with no cementitious binder, lime–hemp composite blocks and unfired earth blocks. To minimise the embodied carbon local materials were sourced and during the construction local labour and professional services were used.

Further information and advice can be found in the sources listed in the references (Forde, 2009, section 10; Franklin & Andrews, 2010/11, 2011; Harris *et al.*, 2009; Structural Engineer Briefing Note (16 March 2010)).

4.5 Construction

4.5.1 Design factors

This section describes decisions that will have to be made in order to ensure sustainable construction. Since the choices of

Figure 4.5 Visualisation Image of the WISE project at CAT. Courtesy of the Centre of Alternative Technology

materials at the conceptual design stage will already have influenced the embodied carbon in the construction of the build we will restrict ourselves here to discussing only additional decisions which can be made in order to make the construction process a sustainable one. These include employing local site workers, using local resources and materials, and methods to reduce carbon emissions from transport to and from site. Not only can these choices reduce the carbon footprint of the build but they also boost the local economy. How a building is to be constructed on site should be built into the structural design. For example, ensuring that the building components will fit on a lorry will mean that the lorry's capacity is maximised to reduce logistics and carbon emissions. In addition, sustainable construction ought to take it a step further and consider how the building can be restored and dismantled to be reused or recycled.

4.5.1.1 Focus 4: Waste

The UK construction industry uses 400 million tonnes of materials every year. Currently only 90 million tonnes are recycled, of which 45 million tonnes become recycled aggregates. It is the UK Government's objective by 2012 to have reduced the waste from construction, demolition and excavation that goes to landfill by 50%. Landfill taxes continue to rise, reaching approximately £48/tonne in 2010. Waste reduction can be achieved through WRAP's five principles of:

- Design for Reuse and Recovery;
- Design for Off Site Construction;
- Design for Materials Optimisation;
- Design for Waste Efficient Procurement; and
- Design for Deconstruction and Flexibility.

A good starting point is to look at WRAP, which is a UK organisation working with the construction industry to help design out waste, and their website is a useful resource (http://www.wrap.org.uk). Through responsible specification and careful design, structural engineers can minimise the disposal of waste to landfill, and thus demonstrate that, through their involvement, project costs can be reduced. On its website WRAP reports that 'WRAP's work with the design team for the Elizabeth Garrett Anderson School, a Building Schools for the Future project in Islington, identified opportunities to reduce waste to landfill by 18 000 tonnes, make cost savings of £303 500, and make embodied carbon savings of more than 2000 tonnes of CO_2 equivalent' (WRAP, 2010).

Further steps can be taken by the use of contracts, specifications, local procurement, monitoring and reviewing site logistics. For more information on site waste management see Structural Engineer Briefing Note (4 November 2008).

4.5.1.2 Significant sustainable wins

1. Appoint 'Considerate Constructors' where there is a commitment to comply as a minimum with best practice site management principles.
2. Measure construction site impacts which involves the monitoring, reporting and setting targets to reduce carbon emissions from site activities such as transport to and from site, water consumption, air pollution, surface water run-off, responsible sourcing of materials and operating an environmental management system.
3. Develop the Site Waste Management Plan in conjunction with the design team.
4. Use of recycled aggregates and secondary use of aggregates in concrete specifications.
5. Reuse of land, contaminated land, ecological survey and biodiversity plans

4.5.1.3 Case study: G Park Blue Planet

The G Park Blue Planet development was one of the first in the UK to achieve BREEAM Outstanding and it has been predicted that it will save £300 000 a year in running costs. There are many contributing factors in achieving a carbon positive site but the most innovative feature of G Park Blue Planet is the use of electro-kinetic road plates set within internal roads to generate electricity from vehicles entering or leaving the site. The project achieved its target of zero waste to landfill through the use of composite panels for the wall construction on a timber frame, using off-site fabrication and using suppliers committed to reducing their own waste. Responsible resourcing has been applied through the use of FSC timber, environmental management system and low volatile paints and it has been stated that 40% of materials were supplied from within 35 miles of the site.

4.6 Operation

4.6.1 Design factors

After going through the design process to deliver any project, handing it over is just the start of that building's life, the beginning of a new phase in its life cycle. To optimise the benefits of creating adaptable and flexible buildings it is recommended to consult with the building users before design starts to appreciate the operational issues so that problems can be solved through the design process. CDM regulations have always asked for an operation and maintenance manual (OMM) manual; however, a building user's guide is a non-technical guide so that the building occupiers can understand the impact of putting up internal partitions when the building operates on cross ventilation or that the floor loading cannot cope with concentrated line loads.

4.6.1.1 Significant sustainable wins

1. Seasonal commissioning of renewable energy sources, building management systems, ventilation, in line with Building Regulations and BSRIA/CIBSE guidelines.
2. Building user guide to include information on the environmental and energy strategy of the building, emergency information, water use, transport and ease of maintenance.
3. Increasing the use of the building by shared facilities and offering the local community a space for clubs and groups and building this into the access, security and operational hours of the building.

4.6.1.2 Focus 5: People

Consideration of people in buildings typically means to the structural engineer: what is the appropriate live load to be used from BS 6399 (BSI, 1996) or what is the existing buildings floor load capacity suitable for change of use? It might be a surprising thing for an engineer to consider but structural engineers ultimately dictate how existing buildings can be reused and how a building's life can be extended. We consciously understand the human impact on the buildings we design. Sustainable buildings require that understanding to be taken a step further to consider not only the dynamics of building physics but also what will happen in the cities of tomorrow. It is quite extraordinary, for example, to think that Victorian industrial sheds have become luxury penthouse apartments for our generation.

4.6.1.3 Case study: National Trust HEELIS

The National Trust's HEELIS building in Swindon won Sustainable Building of the Year in 2007 at the Building Awards and it now has five years' worth of post-occupancy data (Building, 20 September 2007). It is a passively designed building and carbon neutral but it could not maintain that status if the people in the building did not understand the 'layering principle' of clothing and that the building will be cooler in the mornings. In the summer it will be warm throughout the day and in the winter an extra layer might need to be worn. The layering principle also applies to building skins in the winter; more insulation is required throughout the day and in the summer solar gains need to be kept out through external shading and increased air change rates.

The HEELIS design focused on delivering an environment where people want to work and can work through good daylighting, good thermal response and showcases the National Trust's policies. The building was delivered with a limited budget and built for the same cost as a standard building but it only produces 28 kg of $CO_2/m^2/yr$ which is below the BRE benchmark of 32 kg. For further information on the Building Awards see Building (3 May 2012).

4.7 Reuse and demolition

4.7.1 Design factors

It could be argued this should be where the design process starts, in considering how any project is to be demolished so it can be reused and recycled, which comes back to where we started with the site's location and client's brief. However, building reuse represents the biggest structural design problem to be solved – forward thinking would include demountable and temporary structures and the question of how designs can be completely dismantled back into component parts. This is the ultimate challenge and throughout this manual it is a point for consideration.

4.7.1.1 Sustainable urban renewal

Globally there are at least 20 megacities (Pearce, 2006) that have a population of over 10 million each – all with their own unique climates, urban environments and infrastructure. The global population is soon to hit the 9 billion mark (United Nations, 2004) and we are all competing for energy, food, water and resources. A sustainable community will ideally be self-sufficient, or rely on cooperative trading agreements. The idea might sound utopian but it is not new. In *Cities of Tomorrow* Peter Hall discusses architectural theory in urban planning from Le Corbusier's (1887–1965) vision of a 'city of towers' and his design of Unite d'habitation for multi-storey residential use to Ebenezer Howard's (1850–1928) 'Garden City' concept (Hall, 2002). The modern architectural response is captured in Ken Yeang's design of 'Eco Skyscrapers' to increase the growth of high rise buildings in cities and increase the population density in urban areas, so that land becomes available for other uses. Within the urban environment there is an increase in mixed-use developments so that people can live close to work and travel less far, such as Wilkinson Eyre Architects' redevelopment of 20 Blackfriars Road, London. In urban areas we are seeing urban regeneration being promoted with developers like Urban Splash, a rise in allotments along with development of multi-storey green houses such as the

'Vertical Farm Project' so that food can be grown in cities. Sustainable communities are a reality; their future development requires a futuristic vision and a collaborative engineering response to master planning.

Innovative techniques are being developed and legislation is moving forward, and the interfaces between disciplines are likely to merge further especially when considering optimising the efficiency of renewable energy technology.

4.8 Conclusion

Structural engineers can play a significant role in reducing the impact of climate change and depletion of natural resources, not only through compliance with the new standards, but also through doing what we do best – minimising costs by using resources efficiently, solving problems interactively within design teams, having the knowledge and skills to assess and adapt existing buildings, and bringing an open-minded and innovative approach to design.

As professionals, engineers hold the key to the future of the built environment and the role of the professional engineer has been outlined in the UK Engineering Council's *Guidance on Sustainability for the Engineering Profession* (2009). In brief the Engineering Council identifies six roles for the structural engineer:

1. Contribute to building a sustainable society, present and future.
2. Apply professional and responsible judgement and take a leadership role.
3. Do more than just comply with legislation and codes.
4. Use resources efficiently and effectively.
5. Seek multiple views to solve sustainability challenges.
6. Manage risk to minimise adverse impact to people or the environment.

As an industry we are collectively exploring unfamiliar territory but with the historical engineering experience we have amassed we can rise to the challenge and start innovating. To make progress we need buildings that are adaptable to climate change, comply with standards that are not yet written and deliver carbon zero buildings and cities. Skyscrapers similar to the Ken Yeang's 'eco skyscrapers' will soon be commonplace. Cities built with a philosophy similar to Masdar City as shown in **Figure 4.6** and Dongtan will become our urban landscape. These schemes are pushing new boundaries and will set a benchmark for what is to come.

Figure 4.6 Proposed masterplan of Masdar City. Courtesy of Masdar

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Carbon Trust: www.thecarbontrust.co.uk

CEEQUAL: www.ceequal.co.uk

Commission for Architecture and the Built Environment: www.cabe.org.uk

DEFRA: www.defra.gov.uk/sustainable/government/what/index.htm

DIAG: www.diagcomputing.org

Environmental Management Auditing Scheme (EMAS): http://ec.europa.eu/environment/emas/toolkit

Government of Canada: www.g8.gc.ca

Green Building Council of Australia: www.gbcaus.org.au

Global Footprint Network: www.footprintnetwork.org/en/index.php/GFN

G Park Blue Planet: www.gpark-blueplanet.com

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Passivhaus: www.passivhaus.org.uk

Planet-friendly living: www.greenguide.co.uk

US Green Building Council: www.usgbc.org

Vertical farming: www.verticalfarm.com/contact.html

World Wildlife Fund footprint calculator: http://footprint.wwf.org.uk

Wessex Water Building at CABE: www.cabe.org.uk/case-studies/wessex-water/design

WRAP: www.wrap.org.uk

Chapter 5

Taking a through-life perspective in design

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CONTENTS

This chapter brings a through-life perspective to the design process recognising that there are a number of phases in the development and use of a constructed asset; starting with the concept phase, progressing through the design and construction phases, into the operation and use phase, where the proper execution of the previous phases is rewarded by satisfactory through-life performance without major unintended disruptions, costs and environmental impacts being incurred, before the asset is decommissioned and/or demolished at the end of its useful life. Much can be learnt from studying the through-life performance of existing constructed assets, gaining an understanding of what contributes to inadequate performance and lack of durability. These perspectives help explain and respond to the drivers associated with life-cycle cost, value and sustainability issues, which complement and broaden the functional requirements defined for the constructed asset, feeding into the wider design process. Attention is given to the need to create durable constructed assets, with the attendant requirement for a through-life performance plan and a coordinated approach to structural and service life design, construction and associated through-life care processes. Some observations are made upon future challenges and opportunities that are expected to have an influence upon the design process.

5.1 Glossary

ALARP	As low as reasonably practicable: also see SFARP
BRE	Building Research Establishment
BSI	British Standards Institution
CONREPNET	Thematic network on performance-based remediation of reinforced concrete structures
EN	European Standard (European Norm)
LCA	Life-cycle analysis
LCC	Life-cycle cost
LIFECON	Life-cycle management of concrete infrastructures for improved sustainability
MADA	Multi-attribute decision aid
QFD	Quality function deployment
RAMS	Reliability, availability, maintainability and safety
SFARP	So far as is reasonably practicable: also see ALARP
WLC	Whole life cost

5.2 Introduction

5.2.1 General

It is an unfortunate, but inescapable, fact that all constructed assets deteriorate with time, although the rate at which this occurs varies considerably as it is affected by many factors. Whilst most constructed assets will provide satisfactory performance over many decades, there are a significant number of these assets that experience varying degrees of premature deterioration and require one or more remedial interventions to be undertaken, especially where they are located in aggressive service environments. Deterioration can change the performance and the appearance of an asset and it may also adversely affect its functionality under normal working conditions. Deterioration can be particularly worrying when it occurs in locations hidden from direct view.

These difficulties may be compounded by a lack of appropriate maintenance activities and/or preventive interventions, which are defined as works undertaken proactively to reduce the likelihood of future deterioration. Without timely intervention deterioration processes can adversely affect the durability of the asset concerned, and could also diminish its performance and affect its safety.

Amongst other potential influences, the rate and extent of deterioration will depend primarily upon the service environment within which the asset is situated. This may be the overall/general environmental situation (i.e. the macro-climate) or the various local environmental circumstances acting upon discrete parts of the asset (i.e. micro-climate). Thus there are a wide range of issues to be considered at the conceptual and detailed design and construction phases associated with the future management, maintenance and, potentially, the future repair of constructed assets. These issues are concerned with the through-life management of the assets and, as such, are closely linked to matters relating to their service life design.

5.2.2 Learning from the through-life performance of constructed assets

A number of reasons for inadequate through-life performance were described in the 1981 report *Structural Failures in Buildings* published by the Institution of Structural Engineers (IStructE, 1981). This report considered all types of buildings constructed in all materials. The most significant reason for failure was identified as a lack of understanding of the loading conditions and the real behaviour of constructed assets. Ineffective interactions (communication) between different stages and parties in the design and construction process were also cited as an important issue, representing breakdowns in communication within the overall process. The analysis carried out highlighted the significance of communication deficiencies occurring between the concept phase and subsequent phases. The IStructE study did note that the major underlying feature of the structural failures and the in-service deficiencies considered was not so much the issue of any injuries or loss of life caused, important as these consequences are, but the wider economic consequences of the lack of performance of the structures concerned which had important through-life consequences for their owners. The economic consequences of the failure to achieve satisfactory in-service performance were estimated to be much greater to society than those directly associated with injuries or loss of life. Thus the study identified that there were in-service performance issues common to all material types which needed to be addressed.

Whilst most structural problems were discovered during the first 10 years of service, with very few arising after 20 years' service, durability related problems tended to occur at a much later age. Perhaps not surprisingly, the IStructE study (1981) noted that most durability related repairs are apparently made when structures are between about 10 and 50 years old.

Paterson (1984) also studied general building defects, which he did using information collected for the French system of decennial insurance. He used data gathered on 10 000 defects occurring in France between 1968 and 1978 to gain insight into the causes of building defects, which were categorised in terms of the:

- causes of defects, classified on the basis of the cost of repair;
- causes of defects, classified on the basis of their frequency of occurrence;
- causes of design faults, classified on the basis of cost;
- causes of design faults, classified on the basis of their frequency of occurrence;
- occurrence of building defects during the first 10 years of the life of a building.

Paterson (1984) found that design faults and construction faults each accounted for 43% of the cost of repairs (i.e. 86% in total). The importance of good detailing was clear, with 78% of the causes of design faults, in terms of frequency of occurrence, being attributed to this factor. Most building defects (24%) arose in the first year of service. Some 65% of the decennial total of defects arose during construction and in the first three years of service. Overall 52% of the faults arose in the building envelope (in terms of the cost of repair); with 27% relating to external masonry and 25% to cladding and roofing.

Paterson (1984) proposed that engineering expertise and knowledge should be applied to reducing the incidence of building defects and that as the greatest number of these related to the building envelope (i.e. external walls, windows and roof), this would be a sensible place to start. He suggested that the scientific and engineering abilities of engineers should be applied more widely within the 'building team', particularly to the evaluation of the 'value' of new products and systems. The data also implied that greater attention should be given to detailing the weatherproof envelope of the building to reduce the number of interventions required to achieve satisfactory standard of through-life performance.

Whilst a number of the problems identified in the Institution of Structural Engineers' report (IStructE, 1981) and by Patterson (1984) have been addressed by changes implemented over the intervening years, many of the suggested rectification actions are similar to those identified in later studies more concerned with durability (see Matthews and Saunders, 2009, who review a number of these studies). For example, these included the recommendations that a project should be supervised by an appropriately experienced competent person and that design concepts should be similarly checked. Other issues identified included clarity in responsibilities and a focus on quality management. The study also implied that weaknesses in the experience and understanding held within the professional team were more important than general deficiencies present in codes of practice or other consensus guidance.

The above studies suggest that although the technical ability exists to achieve the required level of durability and, in spite of progress having been made over the years, elements of further work still remain to be done. Service life design concepts and, perhaps most importantly, taking a through-life perspective on the performance of constructed assets and the associated cost of ownership, could provide a basis to improve the certainty of creating durable assets – particularly in the case of those which are required to have a longer service life.

Nowadays the consideration of the cost of ownership includes issues such as the sustainability and environmental impacts (i.e. the environmental footprint) of the constructed asset. These pressures are expected to increase further in the future, with drivers such as carbon trading (IStructE, 2010b) influencing the choice/perceived balance between different materials/construction/intervention systems and options. These drivers are also expected to introduce new influences upon the desire to retain existing constructed assets, with their significant quantities of already embodied carbon and environmental impacts, and to expend more effort and resources upon refurbishing and adapting them to extend their useful life.

Reviews by Clarke *et al*. (1997), Jones *et al*. (1997), Matthews and Saunders (2009) and Matthews (2012a & 2012b) of:

(a) the in-service performance of constructed assets, and
(b) previous experiences in seeking to effect change in construction industry processes to deliver better durability and enhanced through-life performance

have also demonstrated that, in addition to defining appropriate technical requirements, there are significant issues to be addressed in respect of people and process considerations. These are commonly referred to as 'soft' issues, as opposed to the 'hard' issues associated with specific technical requirements. People issues include factors such as the number of people available to undertake the task (the resource pool) and their competence. People issues also include factors such as communication, cooperation and coordination, procurement procedures, using experience and applying lessons from the past, etc. Both the 'hard' and the 'soft' issues influence the through-life performance of a constructed asset and, accordingly, have to be addressed together.

5.2.3 Phases through the life of a constructed asset

There are a number of phases in the development and use of a constructed asset. Typically the sequence of events through the life of such an asset is as follows:

- *concept phase* – where the owner's basic requirements and needs are established;
- *design* – usually involving preliminary and detailed design phases;
- *construction* – the process whereby the asset is built;
- *operation and use* – through-life performance and maintenance of its functionality, as well as refurbishment to modernise performance and adapt to revised requirements;
- *disposal* – the process by which the asset is decommissioned or removed.

Figure 5.1 shows a schematic of the through-life processes, the various parties involved and activities associated with the creation of a constructed asset and undertaking a remedial intervention. In this portrayal of the through-life performance of the asset, the operation and use phase has been subdivided into a post-construction service life phase and a post-intervention service life phase.

The way the constructed asset is to be procured, designed, constructed, used and disposed of will have impacts in the various phases of its life. These need to be understood, particularly in terms of any implications there may be for the durability requirements for the asset. Similar influences also arise with the procurement of preventive and remedial intervention works during the life of the constructed asset.

Figure 5.1 also introduces the concepts of a 'Birth certificate' and a 'Re-birth certificate' for a constructed asset. These concepts are defined and discussed further in Section 5.6.13.

5.3 Through-life perspectives – Life-cycle cost, value and sustainability drivers

5.3.1 Introduction

So far the discussion in this chapter has mainly focused upon issues relating to the functional requirements of the constructed asset. These are sometimes referred to as the technical performance requirements. However, there are other wider economic, socio-cultural and environmental factors (sometimes referred to as non-technical factors or issues) that may have an important bearing upon decisions associated with designing and managing a constructed asset.

Clearly design and management decisions need to be undertaken in a holistic way, balancing these different considerations. It is necessary to do this within a suitably broad framework. Previous work relating to life-cycle analysis of constructed assets has provided various frameworks to help define, understand and measure these components. **Figure 5.8** illustrates one classification of these components, utilising the four headings:

- Functional requirements
- Economic and financial requirements
- Societal and cultural aspects
- Environmental considerations

Collectively the later three factors are commonly known as 'sustainability', but definitions of the scope and meaning of this term are perceived to vary significantly around the world. Sustainability and environmental impacts are now key considerations in the world economy and in the creation of the assets and facilities required by modern society. This chapter seeks to bring together considerations of through-life performance, life-cycle cost and sustainability; whilst giving some insight into how they impact upon the service life design and other through-life considerations for constructed assets, such as enhancing and managing value, along with the attendant requirement for durable constructed assets.

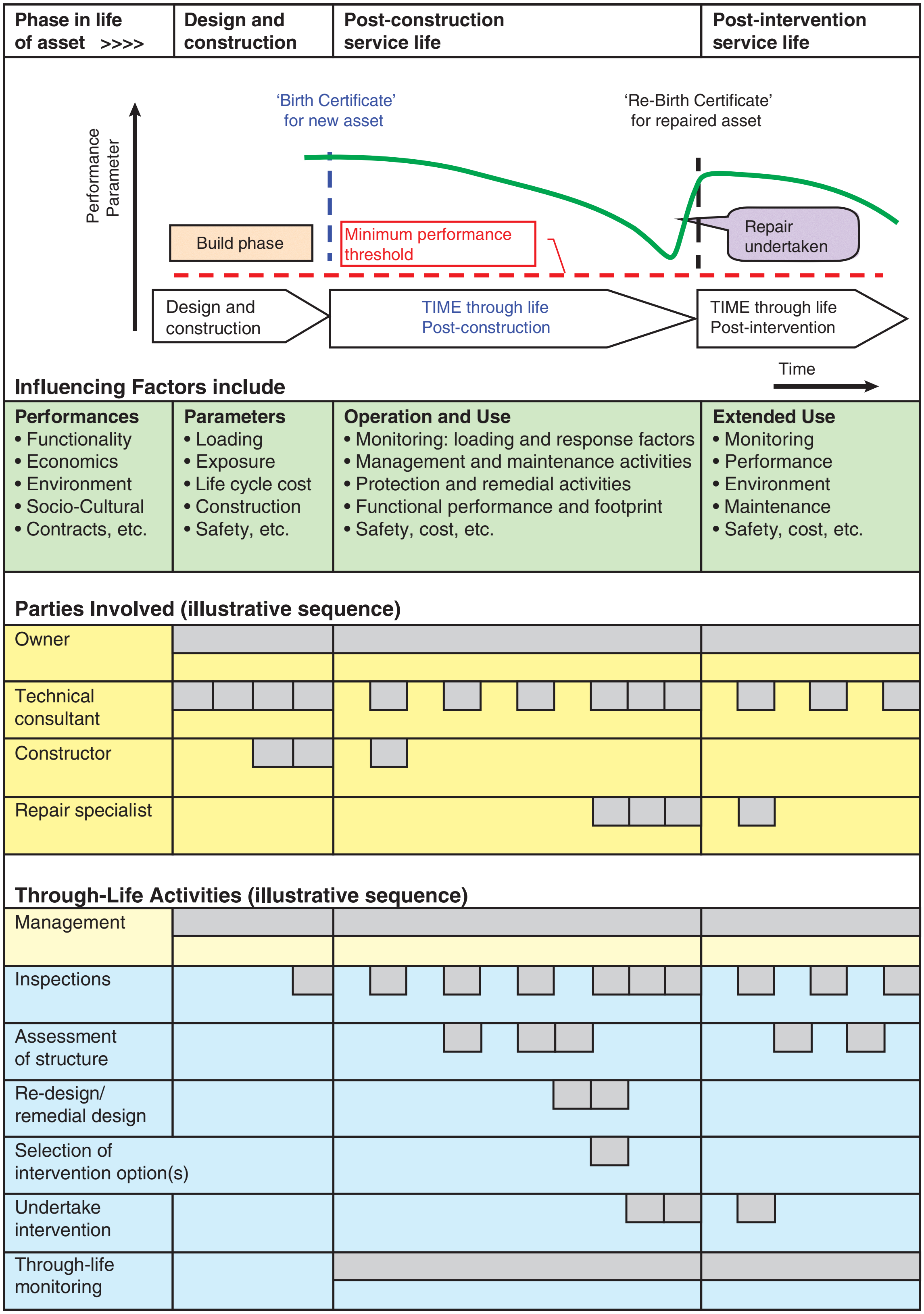


Figure 5.1 Schematic of the through-life processes, parties involved and activities associated with the creation of an asset and in making a subsequent remedial intervention

5.3.2 Service life, life-cycle cost and environmental impact issues

Figure 5.2 illustrates the relationship between the main phases in the life of a constructed asset (on the horizontal axis) and the three factors denoted below relative to the particular phases (time) in the life of a constructed asset, namely:

A. The relative importance of decisions made in different phases in the life of the asset.
B. The relative influence that these decisions can have on the life-cycle cost and environmental impacts of an asset.
C. The potential to add value through the application of value management processes.

Figure 5.2 shows the potentially high influence or impact of factors A to C in the early stages of the life of a constructed asset, and the decreasing influence/impact as the asset passes into the later stages of its life. The potential influence and impact of decisions upon total direct life-cycle cost and environmental impacts of a constructed asset will clearly be greatest when the conceptual development is being undertaken; which is when the least design information is available.

The potential beneficial effect diminishes greatly for decisions made during the later stages of the life-cycle, such that there is little that can realistically be done during the later phases in the life-cycle of a constructed asset to change its performance or influence its life-cycle cost/environmental impact. In addition, attempts to effect changes to the performance of an asset in the later stages of its life will be much more expensive and also probably incur higher environmental impacts from the greater resources required to effect the changes, as portrayed in **Figure 5.3**. Furthermore, changes made at this time will generally also affect the availability of the asset for use and/or impair its functionality while the required works are undertaken. In the case of 'public' assets such as bridges, such actions may also create considerable disruption to users; with the degree of disruption and consequential delay to users depending on the intensity of use. Typically changes in the later stages of the life of a constructed asset will generally provide poor value to the owner and may only achieve a marginal operational benefit.

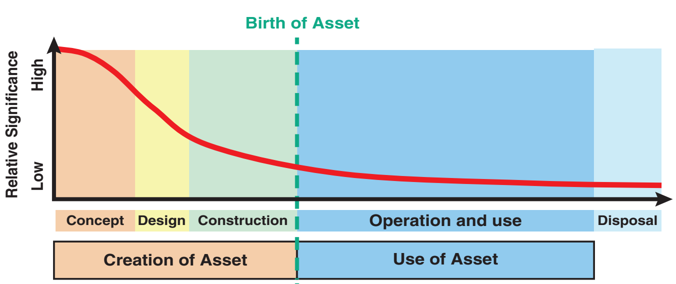


Figure 5.2 Timing of service life decisions relative to (A) their potential impact upon the performance of a structure, (B) their impact on life-cycle costs and environmental impacts, and (C) the potential to add value through value management processes

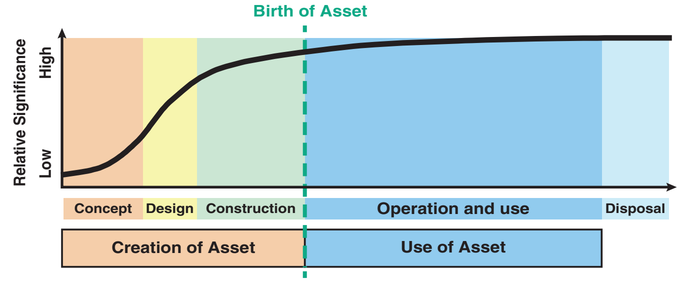


Figure 5.3 Timing of service life decisions relative to (D) the level of knowledge about performance of asset, and (E) the potential cost/environmental impact of any change

Thus these considerations are pertinent in two circumstances, namely:

- during the design and construction of a new asset, and
- when through-life intervention works are carried out on an existing asset.

ISO 15686: Part 5 (BSI, 2008) suggests that up to 80% of the operation, maintenance and replacement costs of a building are influenced by the first 20% of the design process. **Figure 5.7** in ISO 15686: Part 5 presents a curve indicating the scope for life-cycle cost savings during the various life-cycle phases of project for the construction of an asset. The ISO 15686: Part 5 curve defines a similar relationship to that portrayed in **Figure 5.2**, which is sometimes referred to as the 'opportunity curve'.

Total direct life-cycle operational costs could be an order of magnitude (or more) larger than the construction cost, but this depends on the nature of the asset being considered. (**Figures 5.2** and **5.3** consider only direct costs and do not address the potentially much larger business related expenditures associated with the use of the asset, as discussed above.)

For illustrative purposes, design and through-life management strategies for constructed assets may be summarised somewhat simplistically by two conflicting ideologies, namely:

Approach 1. Buy cheap (low initial or first cost) and pay more at a later stage via a higher through-life operational cost resulting in a higher life-cycle cost.

Approach 2. Pay more initially (higher initial or first cost), but gain from a reduced through-life operational cost resulting in a lower life-cycle cost.

However, strictly speaking neither approach is necessarily applicable in all circumstances, as spending money on items that do not contribute meaningfully to extending the service life or improving functionality will simply increase cost without achieving commensurate benefits. Accordingly the investment made has to be appropriate. Thus 'throwing money' at a project by 'gold-plating' the specification is unlikely to mean lower costs later in the life of the asset. However, discounting future costs would confirm whether various investment options brought worthwhile benefits.

Figure 5.4 seeks to illustrate these concepts by showing notional (cumulative) life-cycle costs and environmental impacts; again it is concerned with the importance of early decisions on through-life perspectives in design. The objective is to illustrate the potential value derived from committing significant pre-construction funding to the acquisition of adequate knowledge about the through-life performance of a structure and its component materials and the value of controlling and verifying construction processes in an effective manner. This is necessary to ensure that the (post-construction) behaviour of the structure during operation and use will conform to the required performance levels. In this example, in Approach 1 a repair is required during the operation and use phase of the asset, which incurs additional cost and environmental impacts over those associated with Approach 2.

In **Figure 5.4** the dashed line represents Approach 1 described above; with the solid line representing Approach 2. The difference between the solid and dashed lines symbolises the additional investment or saving being made. Where the solid line is above the dashed line this implies that extra expenditure is being incursolid, such as during planning, design and the stages of construction. Later when the solid line is below the dashed line, this symbolises that a through-life return is being gained on the additional investment made earlier. By the end of the life of the asset, Approach 1 has incursolid a greater overall cost than Approach 2.

A life-cycle cost analysis would more correctly evaluate the overall value being achieved by the two alternative approaches. The use of formal value management processes can help these types of evaluation by providing a framework within which the potential benefits and the costs associated with design and specification decisions are identified and evaluated. Whole-life and life-cycle costing (WLC and LCC) are essentially tools which can be used to enable owners to appraise projects and assist them in making decisions about different projects or options competing for limited financial resources. It enables expenditure to be discounted over time and normalised to a common base year. In essence, WLC and LCC are concerned with providing information relating to three fundamental questions:

- What to do?
- When to do it?
- What will it cost?

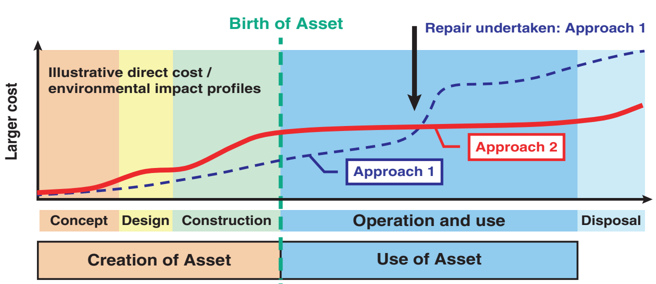


Figure 5.4 Illustrative cumulative direct cost and environmental impact profiles of a constructed asset for two alternative design and operation strategies

The concepts of whole-life cost and life-cycle cost are discussed further in Box 5.1.

Box 5.1 Whole-life cost (WLC) and life-cycle cost (LCC)

WLC and LCC are tools used to appraise projects allowing comparisons to be made about different projects or options competing for limited financial resources. They enable expenditure to be discounted over time and normalised to a common base year. Simplistically LCC might be taken to be limited to consideration of costs associated with the construction and through-life care of the asset, enabling the overall cost of a range of potential construction options to be compared; whereas WLC also considers additional factors which are likely to include the costs associated with the site/alternative sites, as well as matters such as financing costs and revenue stream issues. These techniques are flexible and can, if desired, incorporate many parameters, including those indicated below.

Standard mathematical tools exist and are relatively simple to apply.

Illustrative parameter	LCC factor	WLC factor
Initial capital (construction) cost	Y	Y
Service life/the operational lifetime of the constructed asset	Y	Y
Maintenance, operational and occupancy costs	Y	Y
Demolition or other end of life costs	Y	Y
Costs other than construction cost/ externalities costs		Y
Financial repayment options and revenue streams		Y

A choice is made of the parameters most relevant to the decision to be made and an appropriate discount rate for changes in monetary value in the future is decided. The resultant WLC or LCC calculated is then compared against other competing expenditure in monetary terms at today's value. The WLC or LCC techniques can be applied at various stages throughout the construction process.

The key steps in a simple whole-life and life-cycle costing analysis are likely to include the following:

1. Determine and assess the required service life and what constitutes the end of service life for individual elements and the structure as a whole.
2. Define the study period or period of analysis.
3. For each design option determine the rate of deterioration, the replacement interval and the fraction of each element requiring replacement using appropriate service life forecasting models.
4. Determine the level of deterioration at which intervention is required.
5. Ascribe values to initial capital costs.
6. Ascribe values to maintenance, repair and replacement costs including:
   - Routine maintenance costs (using data for the frequency from Step 3).
   - Costs associated with factors such as traffic management and delays.
   - Costs associated with the loss of use of a structure during planned maintenance/repair.

7. Use the calculated values to estimate the WLC or LCC of the structure. Costs of future work can be discounted to net present values using the expression:

$$\text{Present cost} = \frac{\text{Future cost (after } t \text{ years)}}{(1+r)^t} \tag{5.1}$$

where r is the discount rate and t is the time period in years. The discount rate (or cost of capital) is given by the following expression:

$$\text{Discount rate, } r = \frac{(1+\text{interest rate})}{(1+\text{inflation rate})} - 1 \tag{5.2}$$

Additional aspects of whole-life and life-cycle costing have been discussed by the Royal Academy of Engineering (1998), Somerville (1998), Bourke and Clift (1999), Edwards *et al.* (2000), ASTM E 917–02 (2002), Kesäläinen and Optiroc (2003), the Indian Roads Congress (2004), Bourke *et al.* (2005), BS ISO 15686-5 (BSI, 2008) and PD15686-5 (BSI, 2008).

Figure 5.5(a) Aerial view of Progreso piers
Original 1940s Progreso pier still in service, with the remnants of the 'modern' 1970s neighbouring pier shown ringed

Figure 5.5(b) Original 1940s Progreso pier
This is still in service after exposure to the very aggressive marine environment of the Gulf of Mexico for over 60 years

Figure 5.5(c) Progreso piers
Original 1940s stainless steel reinforced Progreso pier, with the remnant of the 'modern' carbon steel reinforced 1970s neighbouring pier shown in the foreground (Knudsen *et al.*, 1999). Courtesy of Arminox A/S, www.arminox.com

5.3.3 Achieving an appropriate balance between first-cost and life-cycle cost

A practical illustration of achieving balance between first-cost and life-cycle cost is given by the example of two marine piers constructed in Progreso in Yucatán, Mexico on the shores of the Gulf of Mexico (89°W, 21°N). The marine environment at this location is very aggressive with high levels of chlorides present, as well as high atmospheric and water temperatures.

The first pier was built in the 1940s and the designers foresaw that the concrete used in construction could be of 'low' quality and as such unlikely to be able to provide adequate long-term protection to the embedded steel reinforcement from the effects of corrosion due to chlorides in the aggressive marine environment. However, stainless steel reinforcement was specified, which incurred higher construction costs (the solid line in **Figure 5.4**; Approach 2) than if normal carbon steel had been used (the dashed line in **Figure 5.4**; Approach 1). The 1940s pier has performed very well for over 60 years and there has been little deterioration or need for maintenance works – see **Figures 5.5(a, b, c)**. This demonstrates that satisfactory service life performance can be achieved and that repairs are not inevitable in properly designed and adequately maintained structures – even in severe or aggressive environments.

A neighbouring pier was built some 30 years later in the 1970s. In this instance, the structure was reinforced with cheaper carbon steel – which has a much lower resistance to corrosion than stainless steel in the aggressive marine environment. Today the neighbouring pier is totally destroyed, with the only evidence of this newer neighbouring pier being the founding piles sticking out of the sea (see **Figure 5.5(c)** and the dashed ellipse in **Figure 5.5(a)**).

This outcome might be summarised by saying that by the application of appropriate knowledge and expertise, the designers of the original pier were able to utilise low performance concrete to produce a high performance concrete structure.

Clearly spending more on the construction of the 1940s pier has paid significant dividends in terms of reducing life-cycle costs by minimising the expenditure required upon maintenance and repairs. However, the neighbouring pier has to be considered to be an example of a low performance concrete structure. In this case, buying cheap (lower first/direct cost) has resulted in a need to pay much more in the later stages by way of higher through-life operational costs, as the neighbouring pier would need to be rebuilt.

Whilst the above example was for a concrete structure, similar considerations apply to assets constructed in other materials. Furthermore, there are additional factors relating to life-cycle costs that may introduce other considerations into an owner's decision-making process. For example, the cost of capital, as represented by bank interest or discount rates, may alter the balance between various technical options. Lower discount rates favour a longer-term perspective, which would typically involve higher initial costs as portrayed by Approach 2 shown in **Figure 5.4**. Higher discount rates encourage a shorter-term perspective, which puts the focus on minimising initial costs, as portrayed by Approach 1 shown in **Figure 5.4**.

High discount rates have a damaging effect on proper LCC or WLC provisions as it makes it very difficult to justify higher current expenditure, leading to a scenario that 'cheapest is best', even if it implies the need to rebuild the asset in 30 or 40 years. Such an outcome cannot be compatible with the goal of minimising the use of natural resources, as would be generally be expected as society seeks to move towards a sustainable future.

There is also concern amongst some experienced engineers and construction personnel that quality management procedures employed during the design and construction of assets do not always achieve satisfactory product quality and hence durability. An aspect of these concerns is that the overall standard of workmanship being achieved during the construction phase (now commonly referred to as the execution stage) is not always sufficiently high. Such problems can also adversely affect the life-cycle cost and environmental impact of an asset. However, these problems will often have their roots within earlier stages of the project; such as when the design, detailing or material specification tasks are carried out. Special efforts, primarily in the form of pre-planning construction activities and verification of the processes adopted during execution, can be required to overcome these potential difficulties.

In addition, as the above example of the Progreso piers illustrates, there can be a marked difference between employing what in isolation might be taken to be either high or low performance materials and actually achieving a high performance asset which also achieves adequate durability. Thus careful choice and astute specification of materials in design, taking account of construction processes and circumstances, can produce high performance assets; whereas less well-informed choices and decisions can result in the opposite outcome and incur significantly greater life-cycle cost and overall environmental impact.

5.3.4 Achieving through-life value

It can be seen from the above considerations that the cost and perceived 'value' of different service life design strategies can change markedly for different circumstances and in response to the assumptions being made, such as those about the through-life care and management of the asset. These difficulties are compounded by the fact that the perception of value is subjective, with different people applying different criteria and 'weightings' to assess whether something is 'good value'. This highlights the need for a standardised way of evaluating the 'value' achieved. Procedures exist for establishing and managing the achievement of value, which in more explicit terms is taken to be 'value for money'. In this context, value is now commonly and formally defined as the following ratio between achieved benefits and the use of resources (and is often known as the 'value ratio'):

$$\text{Value} \propto \frac{\text{Satisfaction of needs [Both of monetary and non-monetary nature]}}{\text{Use of resources [Cost, time, human, energy and material, etc]}} = \frac{\text{Benefits}}{\text{Expenditure}} \qquad (5.3)$$

The formal procedures for evaluating and managing value in a wide range of circumstances has become known as 'management of value', and this recognises that many benefits are not solely financial in nature. This means that the differing viewpoints of a variety of stakeholders need to be considered when seeking to come to an evaluation of the 'value' of a particular option. The evaluation also needs to consider expenditures that relate to both short- and long-term needs and to take account of the fact that resources to meet the requirements are limited and should be conserved, so that they are utilised in a manner that ensures their availability for future generations. In essence, what these processes are seeking to do is to achieve an acceptable balance between the benefits gained for the expenditure and environmental impacts incurred, considering the potentially diverse range of views and requirements of the various stakeholders involved, for the resources that can be drawn upon.

Clearly this type of evaluation can be conducted in financial terms, such as on the basis of either whole-life cost or life-cycle cost. However, it is also notionally possible that an evaluation could be undertaken on the basis of a number of other criteria, such as the environmental impacts associated with meeting the defined needs. Thus, Equation 5.3 could potentially take a number of different forms, although it is probably fair to say that it is likely to be expressed in a financial format in most current applications.

The current era of financial constraint, coupled with the ever growing pressure on organisations (and hence individuals) in almost all areas of business and service provision to do more with less, means that there is a compelling need to utilise measures that seek opportunities to make significant reductions in expenditure, whilst retaining essential functionality and outputs. These escalating demands are putting ever greater pressure on product and service quality. Management of value processes potentially provides a means of achieving an acceptable balance and ensuring that critical needs are met.

For example, value management processes help organisations achieve such aims by:

- Focusing on functions and required outcomes in a way that clearly states what value means in terms of the short- and long-term needs of the organisation and the end users.
- Supporting decision-making based upon maximising value for money.
- Identifying what the critical needs are and how to deliver them for less.
- Encouraging innovation that is aligned to an organisation's strategic objectives.
- Achieving a balance between investment now and long-term operating expenditure.
- Providing procedures for measuring and evaluating value arising from both monetary and non-monetary benefits.
- Facilitating effective engagement and consultation with all key stakeholders about their differing needs, reconciling their objectives to balance benefits and use of resources and thereby maximise value delivered.
- Being applied throughout the investment decision procedure and/or at all programme and project stages, with its focus changing as these objectives evolve.
- Being tailored to suit the programme or project's environment, size, complexity, criticality and risk profile.
- Encouraging learning from previous experience by creating an audit trail of decisions and actions, enabling sharing of lessons across all projects and facilitating continuous improvement.
- Building a supportive culture with clear roles and responsibilities, thereby providing effective management proportionate to the scale of value management activities.
- Eliminating redundant performance.

Value management is generally considered to be delivered through a seven-stage process as follows:

1. *Defining appropriate boundaries for the project evaluation:* these help establish how value management processes inform the wider business case and supplement other data and information sources.
2. *Gathering input data:* involves obtaining appropriate data and information for the study and includes matters such as the expectations for the value management study, identifying suitable contributors to the study, identifying and understanding stakeholder needs and other project related inputs that are required.
3. *Analysis:* analysing the gathered data and information to provide appropriate input to the management of value study.
4. *Processing of information:* where the value management team use the above derived input to develop bespoke value adding proposals, which may also be innovative.
5. *Evaluating and selecting best value options:* selecting the 'value' proposals that have the most potential for practical and beneficial implementation in the project.
6. *Developing value enhancing proposals:* working up the outline proposals into fully developed recommendations for presentation to decision-making management.
7. *Implementing and sharing the benefits:* developing the plan for implementation of the selected value improvement proposals and for monitoring progress. This stage also involves gathering data and information upon lessons learned and sharing these with others to support the processes facilitating continuous improvement.

The management of value is closely linked with the management of risk, aspects of which are introduced in Section 5.3.8. It is commonly accepted that the uncertainty associated with significant, undefined or uncontrolled risk erodes value; whereas greater clarity in the definition of a project should reduce risk and uncertainty and thereby enhance value. The interaction between risk and value can be approached in two ways: risks identified during value management processes can be fed into the procedures for the management of risk and, conversely, opportunities identified whilst managing risk may be incorporated into the processes for value management. An appropriate balance needs to be sought between risk and value aspects to achieve an optimal solution.

Summarising, the trading-off of additional short-term capital expenditure against future savings in longer-term operating and maintenance costs is a matter of judgement, especially as the level at which these will actually be realised can be hard to predict years in advance. Currently the emphasis is on conserving materials and energy, whilst reducing the overall carbon footprint produced. This is increasing the pressure to invest more in the short term in order to make savings later. Value management processes provide a rational basis on which to do this, with the topic being discussed further in Box 5.2.

In addition to the above, there are other considerations that may have a bearing upon the forms of design or structural solution which are considered to be appropriate in particular situations. One important factor is the relative cost of labour versus the cost of materials. This has a profound influence upon design solutions that are perceived to be optimal or even just simply viable. This is a matter that seems to have received little explicit attention over recent years, perhaps because designers tend to become conditioned by contemporary circumstances within the construction industry in one region of the world. The balance between the price of labour and materials has changed over the years, especially in the so-called 'developed' countries. In these the cost of labour relative to the cost of materials has increased greatly, which has changed the perceived value of different design solutions and the balance achieved in many other facets of modern society. As a result in recent decades there has been a desire to choose design solutions that (implicitly or explicitly) tend to minimise the labour component required in their construction, often at the expense of the use of more material. The labour component can also be that associated with the design stage, as well as that directly related to the construction of the entity concerned.

When materials have been expensive relative to the cost of labour, designers have historically put considerably greater effort into minimising the use of materials to produce an economical design; which was commonly the situation throughout history. For example, the balance between the cost of labour and materials in Uruguay in the 1950s was such that it was economically viable to build simple warehouses using doubly curved brick shells (Thirion, 2010). This situation enabled the famous architect-engineer Eladio Dieste to produce some amazing buildings. Currently such circumstances are more likely to exist in the so-called 'developing' countries of the world.

Thus elaborate structural forms and design solutions become possible even for commonplace types of construction when material costs are high relative to labour costs. In the future as issues such as the amount of embodied carbon that is contained in a material become increasingly central to designers' considerations, the resulting impact upon the 'cost' of construction is expected to encourage designs that either use what are now commonly termed 'low carbon' materials (see Chapter 72 of the *ICE Manual of Construction Materials* (Forde, 2009) for more details on low-carbon building materials) or reduce the use of materials with a high embodied carbon content. These drivers are likely to be amplified should carbon pricing and trading activities become influential factors in the design and construction of new buildings and in the refurbishment or upgrading of existing buildings.

This evolving change in focus is expected to provide not only a new stimulus to designers to meet these challenges, but potentially also more complexity in the way that matters such as value and the suitability of different design options are assessed.

Various facets of the wider framework of sustainability and environment impact related issues that society now expects the construction industry to address are discussed in the next section of this chapter. These complement the economic considerations discussed above. Jowitt (2009) has argued in his 2009 Presidential Address to the Institution of Civil Engineers that a shift needs to take place not just towards whole-life/life-cycle costs, but that we need to go further and move towards whole-life values; with engineers taking a more systems-orientated view of the world in order to address the wider planetary and societal challenges that we face.

Box 5.2 Value management and value engineering

Value, in its broadest sense, is the balance between the benefit delivered to the owner and the cost of doing so. Value provides a measure of whether a project is worth doing and how this can be quantified in business terms (though not necessarily just in financial terms). Value management provides a structured approach to the assessment and development of a project to increase the likelihood of achieving the required benefits for an acceptable cost or use of resources, thereby achieving optimum whole life value for money. The value management process is about ensuring that the right choices are made about obtaining the optimum balance of benefit in relation to cost and risk.

Value management in construction is a continuous process in which all the components and processes involved are critically appraised to determine whether better value alternatives or solutions are available. Value management is also helpful in reducing wasteful processes and identifying less efficient aspects of the design, construction and/or maintenance regimes being considered.

Value management is about enhancing value and not about cutting cost, although this may be a by-product. The principles and techniques employed during the process of developing a project aim to achieve the required quality at optimum life-cycle cost. Value management is important because it enables stakeholders to define and achieve their needs through facilitated workshops that encourage participation of all stakeholders, achieving end-user buy-in and teamworking within an integrated project team. Thus value management activities centre on the identification of the requirements that add demonstrable value in meeting the defined business need. Accordingly the focus of value management is on delivery of the required functionality – whilst recognising time, cost and quality constraints – in order to maximise project value. In some circumstances improved life-cycle value requires extra initial capital expenditure.

Key differences between value management and cost reduction are that the former:

- Positively seeks an optimum balance between quality, life-cycle cost and time (i.e. value).
- Encourages all project participants (including the owner) to work together as an integrated project team to utilise their creative potential to develop better value solutions.
- Provides a structured, auditable and accountable framework for the process.

Value engineering is a part of value management which considers specific aspects of the design, construction, operation and management. Many projects include some unnecessary costs. However, cutting cost without proper analysis of ways of meeting the requirements is likely to lessen value. The removal of wasteful processes and/or practices that incur cost, but do not contribute to meeting the requirements, increases value. Early effort put into developing the project brief pays dividends – see Section 5.6.2. Cutting investment in developing the brief commonly leads to delay and cost overruns later on in the project as a result of changes in requirements and misunderstandings.

Whole-life and life-cycle costing are vital elements of value management as they cover all the costs relating to a facility from project inception through to disposal – they are discussed in Box 5.1 above.

Issues of health and safety, sustainability, design quality, buildability, operation and maintenance and disposal should all be considered during value management reviews and evaluation of options.

In the past inadequate use and understanding of value management and risk management have acted as major barriers to improvement in construction performance. For example, weak risk management on procurement projects has been a common area of concern. Strategic management issues have often been overlooked, with the focus commonly being on technical issues, with a tendency not to pay attention to ongoing risk management throughout the project life-cycle.

Box 5.3 The value of achieving good quality in design

As the built environment affects the lives of many people there is the aspiration that all owners should seek good design for their projects. This needs to take account of the cost and impact of design throughout the life of the project; with owners realising the importance of their role in ensuring that good design is achieved and how this contributes to the delivery of best life-cycle value for money. Clearly it is at the design stage that most can be done to optimise the value of a facility to its end-users, with good design taking account of sustainability and environmental concerns, as well as seeking to minimise maintenance and running costs. Badly designed facilities can be both ineffective and costly.

Thus design quality is about much more than style or appearance. It ensures that key functional requirements of the stakeholders and business are met, whilst considering life-cycle value in relation to maintenance, management and adaptability, health and safety, as well as sustainability and environmental impacts. In order that design quality could be defined and measured the Construction Industry Council (CIC), with support from others, prepared a Design Quality Indicator tool (DQI).

Design quality can be defined in terms of quality elements. For example, for an office workspace, it is suggested in *Creating Excellent Buildings* (CABE, 2003) that the criteria might include:

- Clear open space for maximum flexibility in the layout and use of the space.
- Identity with defined places for entry, reception, work, breaks, drinks and catering services.
- Accessible to all and, where relevant, welcoming to the public and customers.
- A workplace, location and building that represents organisational values.
- Maximising user access to the available views and providing a stimulating outlook.
- Good environmental qualities with local control of lighting, heat, air, etc. in the environment.
- Designing for comfort and ergonomics, particularly in furniture and lighting.
- Use of colour, texture, light and architectural features to enliven the work environment.
- Design for security and safety.
- Design for energy efficiency and future-proofing.

The DQI website (www.dqi.org.uk) assesses building design quality under three main headings:

- *Impact* – relating to character and innovation, form and materials, internal environment, urban and social integration. These factors influence the creation of a sense of place and have a positive effect on the local community and environment. They also cover the wider influence the design may have on the disciplines of building and architecture.
- *Build quality* – relating to performance, engineering systems and construction. These factors influence the engineering performance of a facility and include issues such as structural stability and the integration of health and safety aspects throughout the project life-cycle. They also relate to robustness and resilience of the systems, finishes and fittings.
- *Functionality* – relating to use, access and space. These matters concern the arrangement, quality and interrelationship of spaces and how the building is expected to be used.

Critical success factors for achieving good design quality include:

- A clear brief and sound business case.
- Expert advice as appropriate from knowledgeable owner independent advisers.
- Designers incorporated in the integrated project team with appropriate skills and experience.
- An integrated project team formed at an early stage to allow involvement in all design aspects.
- A suitable site (where choice is possible) with appropriate connections to utilities and services, including transport.
- An effective owner who champions achieving quality in the design process.
- Well-managed design and procurement processes.
- An adequate budget and timescale.

5.3.5 Wider societal sustainability perspective

The economic considerations discussed in earlier sections of this chapter need to be set within the wider framework of sustainability and environment impact related issues which society now expects the construction industry to address. The Brundtland Report's definition of sustainability is widely used: 'meeting the needs of the present without compromising the ability of future generations to meet their own needs' (World Commission on Environment and Development, 1987). However, the simplicity of this definition belies what is a complex web of systems and cycles involving many facets of our society including science, economics, politics, ethics and engineering.

Sustainability cannot be considered solely in environmental terms and there are a number of interacting and potentially conflicting issues and factors that need to be addressed and balanced during the process of designing a constructed asset. **Figure 5.6** illustrates the primary headings under which matters relating to this topic are often broadly grouped. Again these relate to both the construction phase and the processes for through-life management. The *functional factors* are often referred to as being technical requirements, whereas the other topics concerned with *economic, socio-cultural and environmental factors* are sometimes referred to collectively as being 'non-technical' requirements. **Figure 5.6** illustrates one basis for classifying these components. The *socio-cultural and environmental factors* are often referred to nowadays as 'Sustainability'.

There will generally be a number of potentially conflicting issues and factors that need to be addressed and balanced when designing a constructed asset. The weighting of the issues arising may well differ, both in respect of the options available and in relation to owner needs for individual constructed assets. The flexibility of the potential options to accommodate

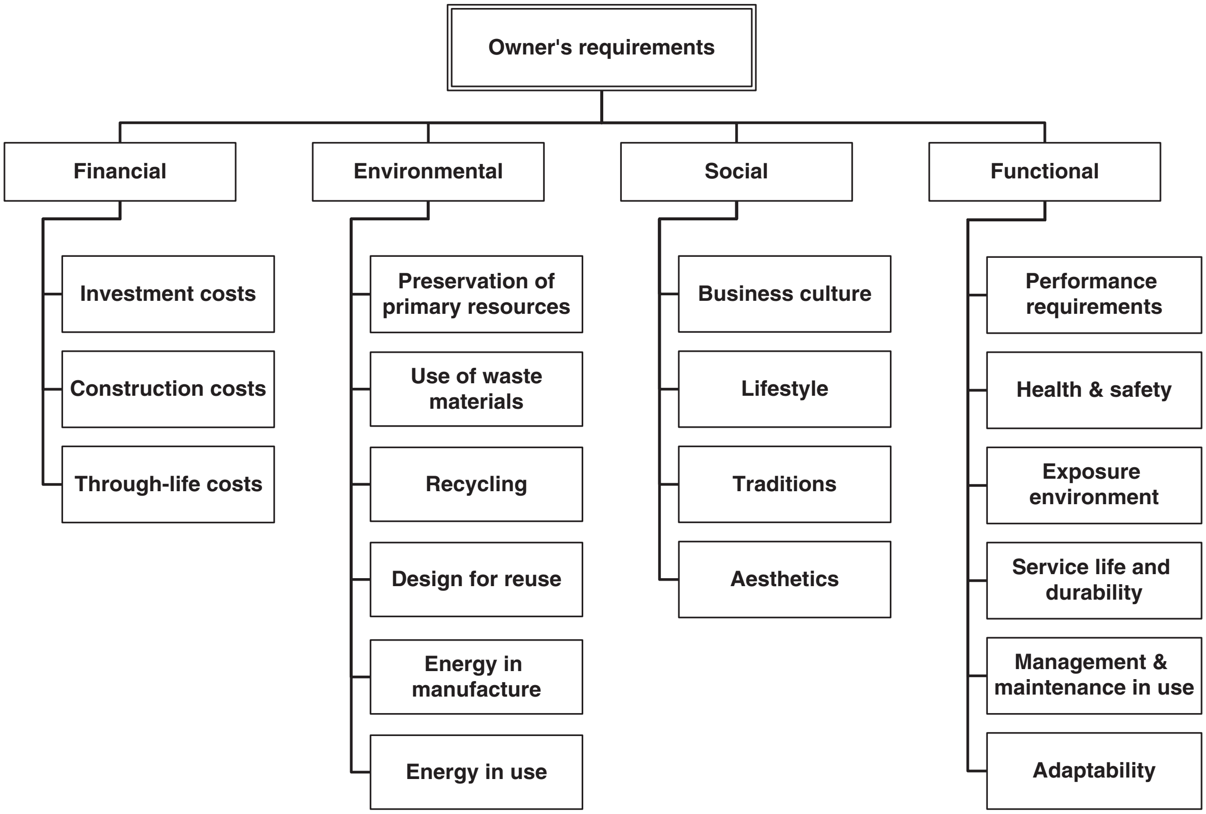


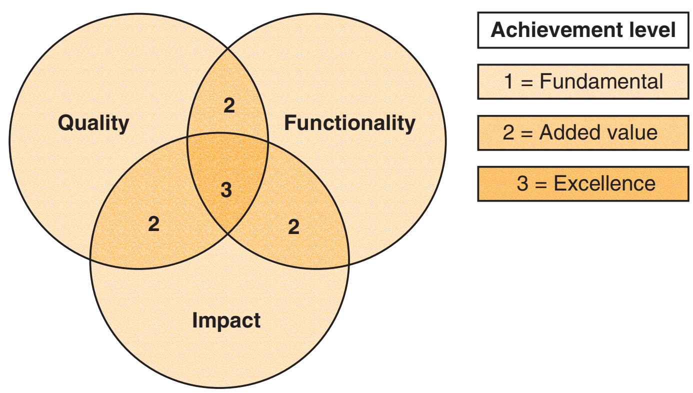
Figure 5.6 One possible breakdown of the components of sustainable construction

changing business need may also be a factor. Issues of maintaining functionality, meeting changes of use or business focus may well be higher order requirements than cost. However, in some cases the issue of affordability or the overriding importance of some non-technical factors can necessitate consideration of what might be considered to be sub-optimal technical (functional) solutions.

Figure 5.7 presents a schematic showing conceptually the goal of combining the three project performance components of quality, functionality and impact to achieve what have been termed 'best value' solutions. The three components relate to the concept of design quality discussed in Box 5.3 above.

This approach can also be applied to the goal of combining the components of sustainable construction, with the objective of being able to achieve an acceptable balance between all the considered factors.

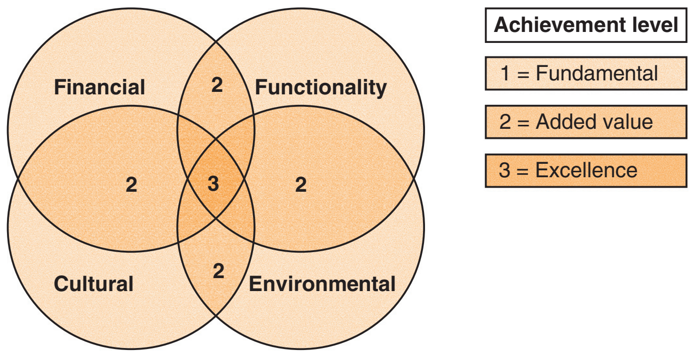
Figure 5.8 takes the above described concepts further and graphically illustrates their application to sustainable construction, with the four components of sustainable construction defined in **Figure 5.6** being adopted. Thus the schematic uses a four-factor representation involving the components of life-cycle cost, environmental impact, functional performance and the cultural contribution of the constructed facility. Interestingly it is noted that the UK Green Building Council (IStructE, 2010a) has suggested that it would like to see more



Where in this context:
Quality = The performance level of the particular item, product, system, etc., including its durability.
Impact = The contribution that the particular item, product, system, etc. makes to the performance of the completed structure/the finished works.
Functionality = How well the item meets the specified requirements and achieves its purpose.

Figure 5.7 Schematic combination of the components of design quality to achieve 'best value' solutions using the three-factor representation style

consideration given to the efficiency of the primary structure which it suggests can be overdesigned by as much as 50% when engineers fail to optimise designs. The UK Green



Where in this context:
Financial = The whole-life cost or life-cycle cost of the particular item, product, system, etc.
Cultural = The contribution and influence that the particular item, product, system, etc. makes to society.
Environmental = The environmental impacts associated with the construction and operation of the particular item, product, system, etc.
Functionality = How well the item meets the specified requirements and achieves its purpose, including consideration of its durability.

Figure 5.8 Schematic combination of the components of sustainable construction using a four-factor representation style

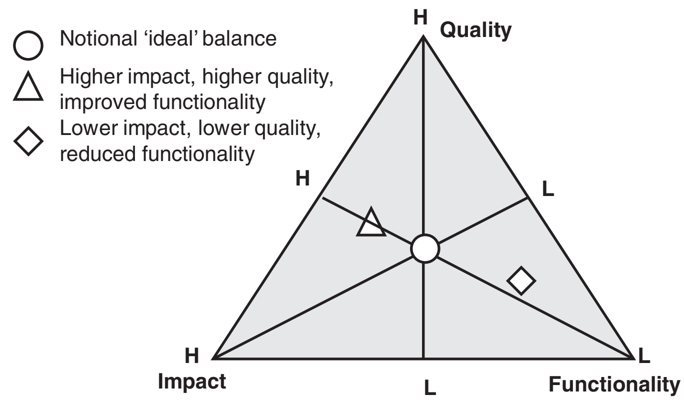


Figure 5.9 Achieving balance between the different components of design quality – Schematic illustrating various compromises in the performance levels attained using the commonly employed three-factor representation style

Building Council's suggestion seems to be compatible with the four-factor representation style presented. Watermeyer and Pham (2011) have proposed a performance framework within which the structural system of both new and existing buildings may be assessed in terms of their fitness for purpose in use and their contribution to sustainable development of the life-cycle of the buildings, which aligns with the concept of having a methodology for the assessment of functionality.

Figure 5.9 illustrates graphically different balances achieved between the components of design quality, with the schematic illustrating various compromises in the performance levels attained in terms of quality, impact and functionality of the constructed facility. This is based upon the commonly employed three-factor representation style portrayed in **Figure 5.7**.

If the four-factor representation style presented in **Figure 5.8** were to be illustrated in the same way a four-leg radar type plot might notionally be employed.

There are a number of environmental assessment methodologies available that consider a range of factors contributing to sustainable construction within formal scoring systems which give an overall evaluation – some of these are discussed in Section 5.3.7.

Formal scoring and evaluation systems require a method of weighting and combining the various influencing factors, as well as evaluating the potential options. The CONREPNET project (Matthews *et al.*, 2007) and the LIFECON project (see below) examined a number of tools supporting option evaluation and decision-making in such circumstances. These tools include various methods for multi-parameter decision making, such as:

- Multi-attribute decision aid method (MADA) (Lair *et al.*, 2004).
- Quality function deployment method (QFD) (Lair *et al.*, 2004).
- Quality function deployment (QFD) in combination with reliability, availability, maintainability and safety (RAMS) evaluation (Söderqvist and Vesikari, 2003).

As part of their work, the LIFECON project partners also developed an overall reliability based methodology for the through-life management of concrete structures (Sarja, 2004).

It is expected that in the future it will become increasingly important to recognise the influence that the emission of greenhouse gases, as well as the impact of regulatory instruments such carbon taxes or carbon trading, may have upon the evaluation and selection of different design options. In the past the amount of 'operational carbon' associated with the daily energy requirements of buildings and some other forms of constructed assets have been relatively high, perhaps in the order of 85% of the life-cycle total.

The balance between operational and embodied carbon is likely to change over time as improvements occur in the energy efficiency of buildings and other assets. This means that as operational carbon reduces in future design solutions, the embodied carbon component of the life-cycle total is expected to become proportionately larger and a more important factor in the future. Such issues are expected to provide new stimuli to designers to create different design solutions, but a through-life perspective in design will remain important in these considerations. There are similarities with the observations made earlier about the influence of the balance between labour and material costs upon what are perceived to be optimal design solutions, which of course reflect the constraints and requirements applicable at the time of design.

5.3.6 Service life and sustainability considerations

The concepts and processes required to balance the various and potentially conflicting components of sustainable construction have collectively been termed 'lifetime engineering'. Research in this field is leading towards the development of an integrated, holistic approach to the design and management of buildings and structures which takes account of all aspects of design and construction practice, including recognising the benefit of integrated teams and supply chains. This type of approach is seeking to encompass both cost-driven and sustainability-driven initiatives aimed at delivering sustainable construction at an affordable life-cycle cost. It should also be recognised that through careful and intelligent choices, good design solutions for constructed assets that achieve a lower overall environmental impact can potentially be made even with materials that have a relatively high environmental impact per tonne. Voo and Foster (2010) present several examples where such a design approach produces a more sustainable solution. They compare the use of ultra-high performance concrete, which has a significantly higher environmental impact per tonne, with conventional Portland cement concrete, firstly, for a 40 m span highway bridge in Australia and, secondly, for a 1.5 m high retaining wall to a monsoon storm water drain.

Service life design concepts (see Boxes 5.4 and 5.5) can be key facilitators for the delivery of sustainable construction. When used in conjunction with appropriate conceptual and detailed design, in association with properly planned and delivered execution, it can help meet the owner's needs in terms of the performance of the structure over its lifetime in an economic manner. An important facet of a sustainable construction strategy is the quality of construction and whether this is appropriate for the envisaged service life requirements of the structure. Failure in sustainability terms can arise from both over- and underdesign. For example, overdesign might occur where more or higher quality materials have been used than are needed to meet the design requirements. It might also arise through underdesign or as a result of poor quality materials, design or workmanship which results in the need for premature repair or replacement.

Significant progress towards these overall objectives can be made by making relatively simple, but intelligent, decisions about design and detailing, taking account of the characteristics of the structure's service environment. Overwhelmingly these considerations involve the control of water and limiting the access of moisture to the structure. It is suspected that considerable improvements could be achieved simply by getting more of the 'basics' correct, without recourse to special materials or sophisticated analyses of failure mechanisms.

A service life design approach provides a basis for addressing many aspects of these issues. The key elements of the service life design system are reproduced in the list below. These items act as a reminder that durability problems can arise as a result of deficiencies which occur in any one of a number of stages throughout the design, specification and construction process.

- Owner brief.
- Assessment of environmental loads.
- Definition of required performance under environmental loads.
- Development of project specification.
- Conceptual and detailed design phases.
- Execution of works.
- Through-life care and maintenance.

Figure 5.10 presents a simplified schematic diagram of the overall design and construction (execution) process for a constructed asset, with associated links to the service life design process being shown on the left-hand side of the diagram and construction process sustainability related activities being shown on the right-hand side of the diagram.

In this context items such as 'loads' are intended to relate to both physical loads and actions associated with structural design and also to the environmental loads associated with service life design/durability considerations. The diagram is intended to illustrate the interactions between the structural design and service life design processes. All the above activities are carried out in parallel and typically proceed in an iterative manner.

Clearly the actual service life design process is somewhat more complicated than the above diagrammatic representation portrays. **Figure 5.17**, which presents a more specific representation of the service life procedure for a concrete structure, puts a strong emphasis on the owner (client) brief and conceptual design stages, whilst embracing structural design. Execution, construction quality and through-life care and maintenance in use are crucial if the intended performance is to be achieved in practice. As noted previously, these factors need be considered at the conceptual design stage. The importance of good communication up and down the supply chain and across the service life design process is also highlighted in **Figure 5.18**.

Figure 5.10 also makes reference to a durability assessment review procedure, which is an auditing procedure applied throughout the service life design process. This type of additional activity, linked to wider quality assurance goals, would be intended to improve the likelihood of achieving the owner's and user's requirements in terms of durability, sustainability and functional performance for the anticipated structural/life-cycle cost and environmental impacts.

5.3.7 Overview of environmental assessment methodologies

Environmental assessment methodologies have to encompass the complexity of the relationships that exist between different sustainability issues, whilst employing practicable means of evaluating the environmental and other aspects of the 'impacts' arising from a building or other constructed asset. Many of these issues are interrelated, which adds to the complexity of making the desired assessment.

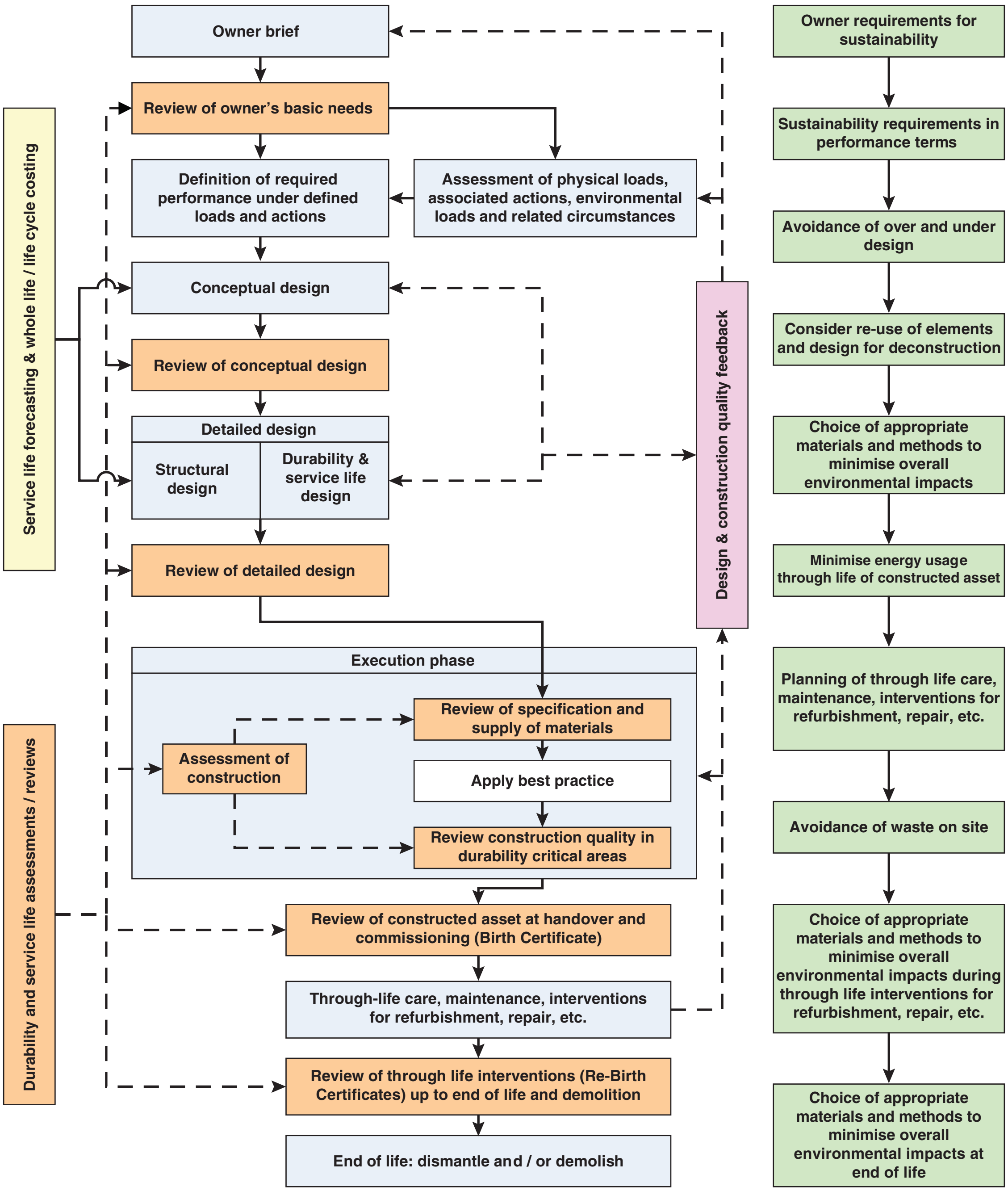


Figure 5.10 Schematic diagram illustrating the overall design process with associated links to service life design and sustainability-related initiatives (Nixon, 2002)

There are aspects of these impacts which are associated with the construction of the building or asset concerned (i.e. the embodied impacts), and also those aspects which arise from the through-life performance of the building or asset (i.e. the operational impacts). For many contemporary buildings the through-life performance characteristics are the dominant influence. For example, in buildings the operational energy used has typically had the greatest bearing upon the total through-life environmental impacts incurred, with the operational energy derived impacts amounting to perhaps 75–85% of the overall through-life environmental impacts.

However, in future operational energy usage is expected to be greatly reduced in response to government legislation and other drivers encouraging a move towards 'low operational carbon' buildings. In such circumstances, the environmental impacts associated with construction (i.e. the embodied impacts) would become the primary contributor to the total through-life environmental impact. In turn this would be expected to bring a greater focus in design on issues such as the optimal service life and on other through-life perspectives. Accordingly, it is necessary to understand the environmental assessment methodologies which are employed, how they operate and influence design decisions. It is especially important to recognise the potential bearing they could have, either beneficial or adverse, on aspects of through-life behaviour of the building or asset concerned.

The key environmental issues are listed in Table 1 of BRE Report 502 *Sustainability in the Built Environment: An Introduction to its Definition and Measurement* (Atkinson *et al*., 2009), which also makes observations about the importance to humans of the key environmental issues listed.

The practical ways of addressing sustainability measurement and delivery which were developed by the pioneers of sustainability assessment in the built environment focus on the key issues in terms of their economic impacts, environmental impacts and social impacts.

The resulting environmental assessment methodologies that have been developed have sought to achieve a balance between simplicity in the operation of the methodology with clarity, transparency, accuracy and the validity of the procedures involved with respect to the underlying scientific issues.

The environmental assessment methodologies have typically been developed with extensive stakeholder engagement and peer review. This has not been a simple task and as understanding of the scientific issues improves, it is expected that these methodologies will continue to evolve with the goal of reconciling and achieving a balance between all of the key issues, which are inextricably linked. In essence, society and the construction industry are still at the start of this journey. However, it is clear that these methodologies and tools are already having a significant influence on design procedures and the design solutions that are being adopted. As noted above, as measures to reduce the through-life operational energy used achieve their objective, the balance will change between the environmental impacts derived from operational activities and those associated with the construction (i.e. embodied impacts).

The UK has played a leading role in developing the principles of sustainable construction since 1990, when the BRE Environmental Assessment Method (BREEAM – see www.breeam.org) was launched. BREEAM, the world's first environmental assessment method for buildings, was the result of many years of collaborative development of codes, standards and toolkits by a network of organisations working with the UK government, agencies, industry and universities.

Over the years the regulatory and voluntary framework for sustainable construction in the UK has evolved and developed greater sophistication. Details of the main regulatory requirements relating to sustainability in the UK and the voluntary codes and standards that lie alongside them are listed in Table 2 of BRE Report 502 (Atkinson *et al*., 2009), which presents these mapped onto the key stages and activities in the construction life-cycle. All of these factors have had a bearing upon the design solutions adopted, and these influences can only be expected to increase in the future.

In addition to the growing BREEAM family of standards and related tools (see **Figure 5.11**), other significant environmental assessment standards and tools used in the UK include:

- CSH – Code for Sustainable Homes (refer www.planningportal.gov.uk).
- *The Green Guide to Specification* (Anderson *et al*., 2009) (www.thegreenguide.org.uk).
- CEEQUAL – the Civil Engineering Environmental Quality Assessment and Award Scheme (www.ceequal.com).
- DQI – the Construction Industry Council's Design Quality Indicator (www.dqi.org.uk; see also Box 5.3 above).

These are supported by other initiatives, schemes and standards such as those dealing with environmental profiling of materials and construction products using life-cycle analysis (LCA) data, and those concerned with supply chain management and responsible sourcing. These include:

- BES 6001: *BRE Environmental and Sustainability Standard: Framework Standard for the Responsible Sourcing of Construction Products* (BRE Global, 2008).
- BES 6050: *Methodology for Environmental Profiles of Construction Products: Product Category Rules for Type III Environmental Product Declarations of Construction Products* (BRE Global, 2010).
- BS 8902: *Responsible Sourcing Sector Certification Schemes for Construction Products Specification* (www.bsi-global.com).
- FSC – the Forest Stewardship Council scheme (www.fsc.org).

Internationally a number of other environmental assessment methodologies and standards have been or are being developed. These include

- BREEAM International (www.breeam.org).
- Green Globes (www.greenglobes.com).
- LEED – Leadership in Energy and Environmental Design (www.usgbc.org/leed).
- Green Star (www.gbca.org.au/green-star).
- CASBEE – Comprehensive Assessment System for Building Environmental Efficiency (www.ibec, www.jp/casbee/english and www.greenbuilding.ca).
- BS EN ISO 14001:2004: *Environmental Management Systems* (www.bsi-global.com).
- European Commission Mandate M350: Integrated environmental performance of buildings (www.cen.eu).

The above are complemented by a range of sustainability related initiatives variously aimed at influencing the economic,

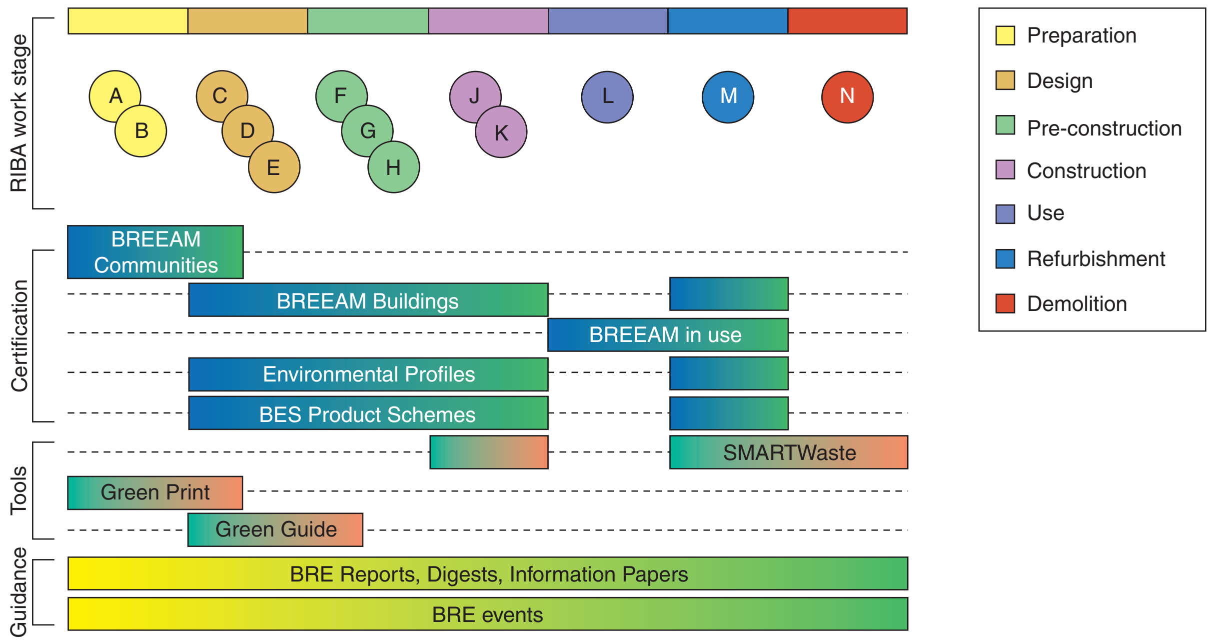


Figure 5.11 The BREEAM family of standards and tools (Atkinson *et al.*, 2009). Courtesy of BRE

environmental and social performance of organisations or with the goal of moving the global built environment onto a more sustainable basis. These initiatives could be influential upon the owner of the constructed asset, have a bearing upon their objectives and aspirations for the through-life performance of the asset in question.

Programmes, organisations and/or initiatives active in these areas include:

- World GBC – World Green Building Council (www.worldgbc.org).
- UNEP – United Nations Environment Programme (refer www.unep.org).
- SBA – Sustainable Buildings Alliance (www.sustainablebuildingsalliance.org).
- GRI – Global Reporting Initiative (www.globalreporting.org).
- World Wildlife Fund's One Planet Future scheme (www.wwf.org.uk/oneplanet).

In addition, the development of company tools, such as Arup's SPeAR® (Sustainable Project Appraisal Routine), reflects not only the growing construction industry interest and participation in sustainability, but also contributes to raising awareness of assessment procedures and to the wider debate about selecting appropriate design solutions.

BRE Report 502 (Atkinson *et al.*, 2009) provides brief descriptions of the various environmental assessment methodologies, standards, tools and sustainability related initiatives. It also discusses common features of environmental assessment tools.

It is also interesting to reflect on how environmental assessment methodologies might potentially evolve in the future and whether there will be significant evolution in the basis of and methodologies for assessment in light of the implications of the further advance of computer based graphical design tools and building information management tools. It is expected that these could have considerable implications not only for the processes of design and operation of constructed assets, but also for the way that environmental assessment might be undertaken. For example, the potential automation of these procedures and their incorporation in graphic interface design tools giving 'real-time' indications of environmental impacts and life-cycle cost implications as design work proceeds could be very powerful. See Chapter 4: *Sustainability*, for a further discussion of these matters.

5.3.8 Risk management

Risk is a term used in the context of exposure to some form of adverse or unpleasant outcome (i.e. a hazard). Typically in human terms this is in relation to the danger of injury or death through the occurrence of an undesired event. However, it also applies in a wide range of other circumstances, such as some form of deterioration, damage or harm to an asset, as well as in terms of an economic or some other form of loss.

Risk is defined as the combination of the likelihood of occurrence of a particular hazard and the magnitude of the consequences thereof. Thus risk is a measure of the estimated magnitude of a hazard, bearing in mind its likely rate of occurrence. In engineering situations the concept of risk is typically expressed by an equation of the following form:

Risk *(consequence/unit time)* =
Frequency *(event/unit time)* × Magnitude *(consequence/event)*

Thus there are a wide range of types of risk, such as business, commercial or economic risk, life safety and injury risk, etc. These have varying degrees of relevance to the different aspects of through-life design and how these perspectives apply to the circumstances of a particular project.

In the context of a commercial project risk is commonly viewed as uncertainty of outcome, whether that has a beneficial effect, in terms of creating a positive opportunity, or has an adverse (negative) impact. Generally some amount of risk-taking is inevitable, whatever the nature of the project. Commonly the value to a business of the potentially beneficial outcome makes it worthwhile to consciously and deliberately accept a certain degree of commercial risk. Risk management is the process of actively evaluating, accepting and controlling some risks; whilst seeking to minimise or exclude some other risks. The process includes all activities required to identify and control the risks associated with the selected project option.

Successful risk management requires senior management commitment, ownership and understanding of the processes involved, as well as an active risk analysis and management regime which is reviewed regularly. Such processes work most successfully in a constructive 'no-blame' company or project culture. In this the organisation's or project's attitude to risk has a very great effect on how matters are approached. If the organisation or project team has a very low tolerance of risk (i.e. is risk averse), the objective could well be to avoid 'failure' of any kind. Conversely, if the approach is more 'entrepreneurial', the desire to 'succeed' will typically encourage participants to be more innovative, to take more risk where this is appropriate and to make more effort to monitor and manage the recognised risks associated with a particular project.

As risk profiles change with time, management of risk needs to be an ongoing process carried on throughout the life of a project or organisation. The goal is to develop risk management plans that allow risks to be dealt with quickly and effectively should they arise.

Although these considerations might commonly be divided into two main phases relating to (1) the design and construction of the asset and (2) its operation and use, there clearly are strong linkages between these phases in the life of the asset that need to be taken into account and managed appropriately.

Risk management is a systematic approach which is used to avoid, reduce or control risks. The course of action followed is to assess uncertainty by identifying and assessing hazards, understanding, acting on and communicating risk issues. The goal of risk management is to protect the owners and users from various factors such as economic losses, injuries, etc. There needs to be a balance between the cost of managing risk and the benefits expected from taking the risk.

In these processes there clearly are numerous aspects to be considered including a need to:

- Establish the nature of the hazards involved.
- Establish what hazard scenarios should be considered.
- Make an evaluation of the likelihood of occurrence and the potential consequences.
- Consider the potential outcomes (consequences).
- Consider whether there are particular classes of risk, such as those imposed by statutory obligations under law or associated with specific business objectives.
- Clarify whether the risks change with time.
- Identify the internal and external stakeholders involved.
- Clarify when a risk should be controlled or reduced and how can this be done.
- Define which risks are to be carried and by whom, together with the criteria which should be used in this process.

Thus once a hazard has been identified and quantified in some manner, the decision has to be made whether the associated risk can be accepted or not. Eliminating hazards is the prime objective and then, if this has not proved possible, to invoke the steps involved in reducing risks from the remaining hazards.

If risks are considered to be too large for direct acceptance, the standard approach is to look for adequate counter-measures. When planning counter-measures, it is first necessary to recognise possible hazards. The aim is to detect those events or processes where a significant benefit can be obtained from a proportionally small input effort. Possible counter-measures can be technical or administrative and can fall within the following strategies:

- *Avoid* the risk by removing the hazard by changing the concept or the objectives.
- *Reduce* the cause of the risk by modifying or altering the nature of the hazard or by reducing the likely frequency of occurrence or by modifying the potential consequences as far as may be possible in the circumstances.
- *Control* the risks by employing monitoring and communication techniques – which in the case of engineering systems might be by being vigilant and through the use of suitable alarm systems, inspection regimes, etc.
- *Overcome* the risks by providing adequate strength or capacity for the worst credible loading or performance requirement.

Risk assessment for activities in the design and construction phases needs to reflect the particular tasks being analysed and the wider circumstances which might exist. Accordingly each is situation specific. Risk assessment for the planned life-cycle of the system under consideration involves consideration of:

- the potential frequency/probability of occurrence of the hazardous situation;
- the severity/consequence of the worst possible outcome arising from the hazard.

There are two approaches to undertaking risk analysis and assessment, these are:

- qualitative risk assessment;
- quantitative risk assessment.

Figure 5.12 provides an overview of risk analysis, showing schematically the factors associated with both of these approaches.

One aspect of the tolerability of risks from a UK perspective is illustrated in **Figure 5.13**. This indicates the levels of annual probability of death of an individual at which the risk is considered to be intolerable/unacceptable (10^{-4}) and the level at which the risk is considered to be broadly acceptable (10^{-6}), with the ALARP/SFARP (HSE, 1988/1992, 2002) zone of tolerable risk existing between these two boundaries, where:

ALARP = *As low as reasonably practicable*.

SFARP = *So far as is reasonably practicable.*

Matthews and Reeves (2012) provide a more detailed discussion of risk issues and the related aspects of hazard identification, the quantification of the probability of occurrence of particular hazards, along with their evaluation and treatment in the context of the through-life management of large panel system built dwelling blocks. Their considerations are particularly focused on life-safety issues.

Many others including Blockley (1980), Hambley and Hambley (1994), Menzies (1995, 1998), Schneider (1997), Melchers (1999) and Evans (2005) have discussed the issues of hazards, hazard identification, risk evaluation, safety appraisal criteria and structural safety. See Chapter 3: *Managing risk in structural engineering*, for a further discussion of these issues.

5.4 Creating durable constructed assets – The need for a through-life performance plan

As part of the conceptual design process, a through-life performance plan can be developed which provides a simple outline of the design service life requirements of the various elements of the asset which is to be constructed. These include non-structural elements of the structure or asset concerned. The principle is illustrated in **Figure 5.14** for the example of a bridge. In this case, items such as the foundations, piers and abutments are required to last the life of the bridge (e.g. nominally 120 years for a UK bridge) whilst the other components are intended to either be maintained and/or replaced at shorter periods within the design life of the bridge. This approach, when taken in conjunction with an appropriate understanding of the aggressivity of asset's service environment, helps the early development of a holistic view of the durability related issues.

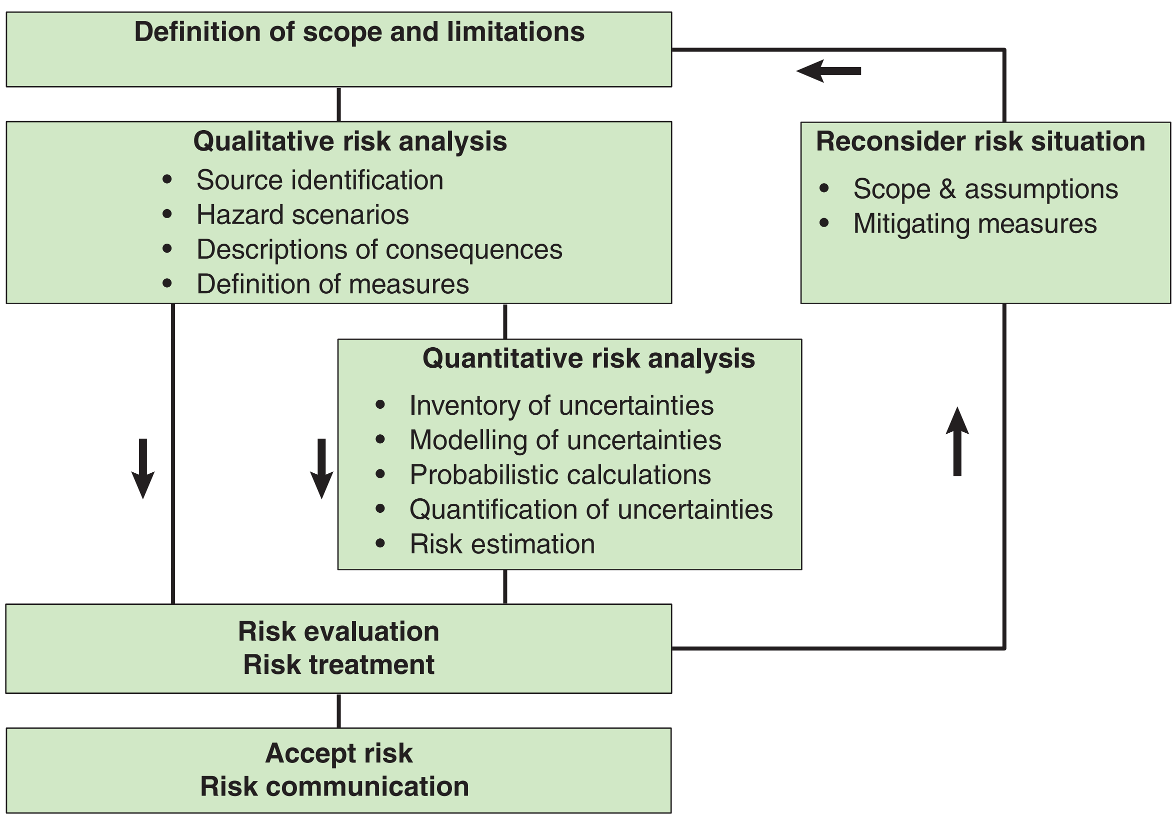


Figure 5.12 Overview of risk analysis. Adapted from BS EN 1991-1-7. Permission to reproduce extracts from BS EN 1991-1-7 is granted by BSI

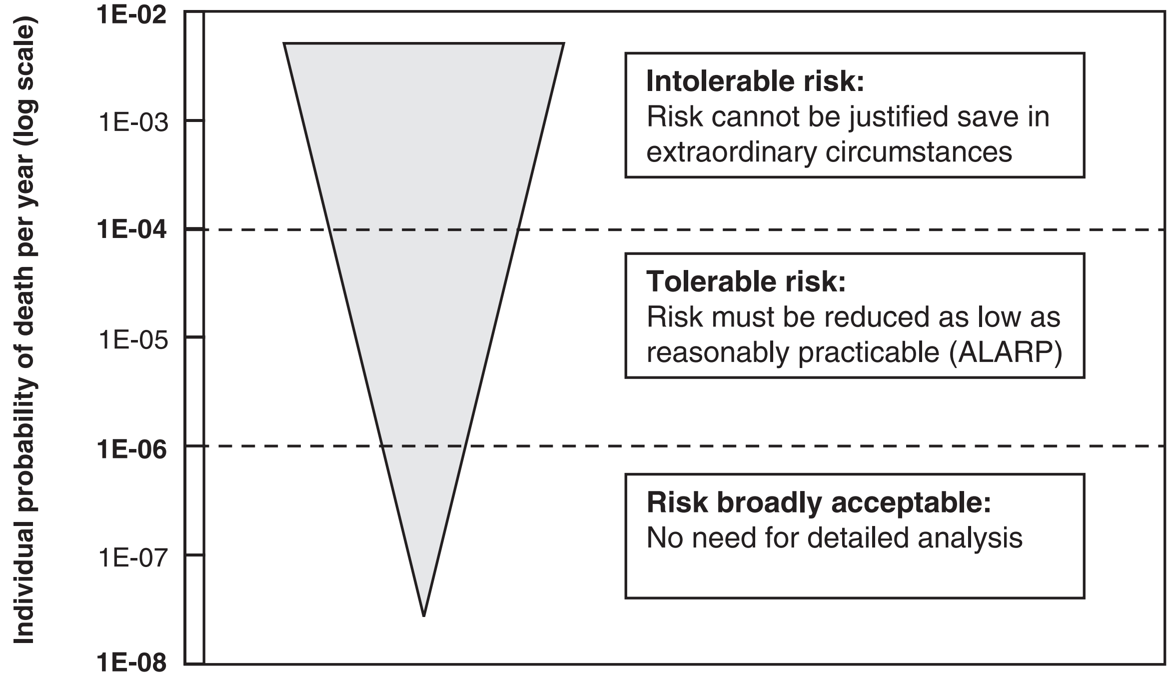


Figure 5.13 Life safety of individuals – the tolerability of risk (TOR) framework (HSE, 1988, revised 1992) © Crown Copyright 1992

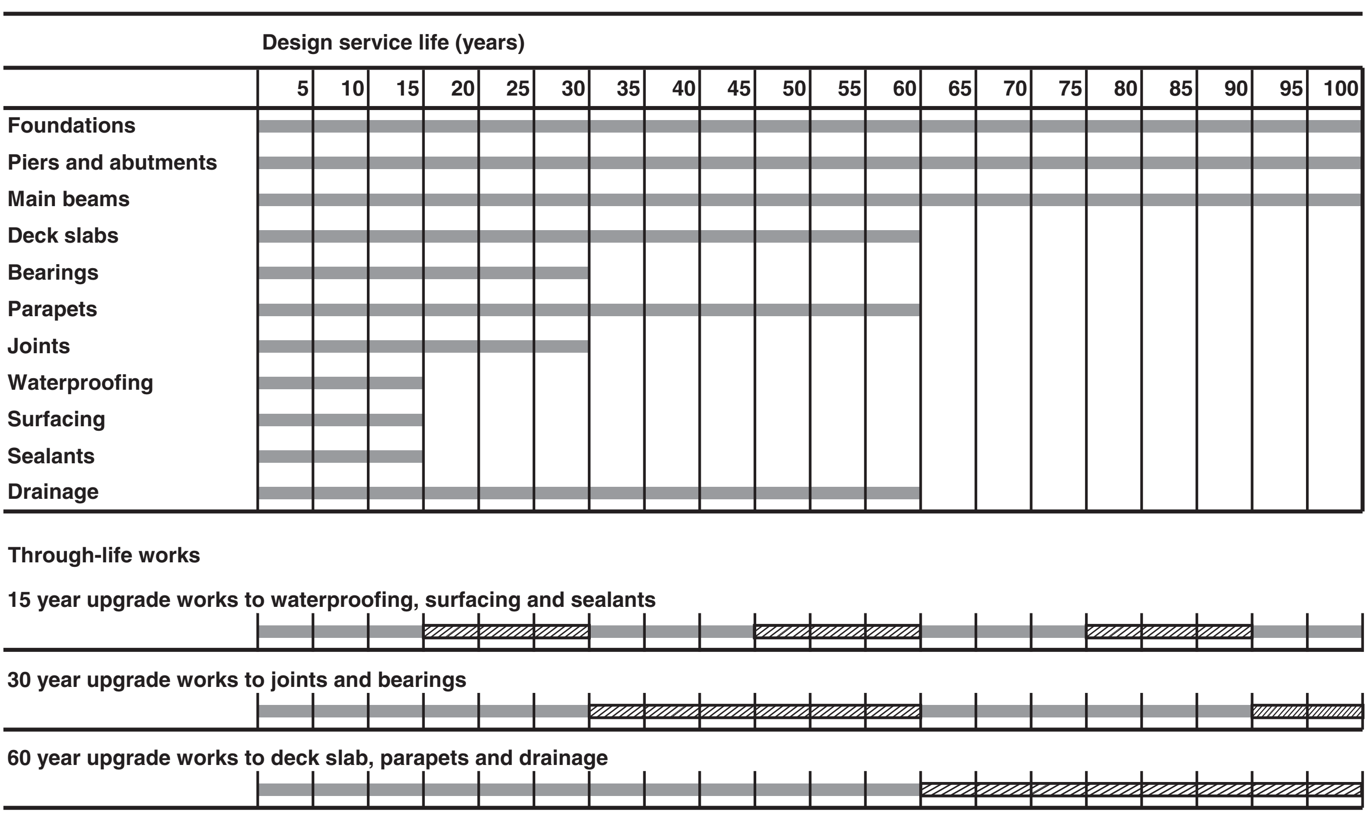


Figure 5.14 A notional through-life life performance plan for elements of a bridge (after BRE IP3/06 – Part 1: Nolan *et al.*, 2006). Courtesy of BRE

The through-life performance plan approach provides a record of these early decisions and a means to:

- record the design intent with regard to service life throughout the evolving design;
- check the anticipated maintenance requirements against the owner's expectations for maintenance and functionality;
- record any maintenance related decisions taken during the service life design process;
- record those aspects of the design intent which need to be passed on to the owner/user of the structure, perhaps by way of a 'birth certificate' or an owner's/operator's manual for the constructed asset.

Approaches to protecting the structure identified at the conceptual design stage will need to be further developed as the design process evolves into the detailed design. The approach taken will depend on the nature of the asset to be constructed and its service environment. In many instances the provision of basic resistance measures, in combination with addressing potential construction quality and buildability issues, will be sufficient to ensure that the asset is adequately durable.

However, there may be situations where a more rigorous approach to service life design will be required, for example, where:

- The service environment is particularly harsh.
- The functionality of the asset or its component parts cannot be disrupted.
- A long service life has been requested by the owner.

Figure 5.15 presents a decision matrix to assist the designer in identifying those elements that are at potential risk from premature durability failure and illustrates their importance to the function of the asset. This assessment requires characterisation of the service exposure environment and the assessment of the criticality of the component parts of the asset. An iterative approach can be used where significant risk/consequence combinations are identified, allowing the designer to alter the form and detail of the structure to achieve sufficient durability for the asset, especially for the critical parts. The approach can be applied in outline at the conceptual design stage, being refined in the subsequent detailed design stage.

In BRE Information Paper 3/06 – Part 3 Nolan *et al*. (2006) provide an illustration of how these concepts may be applied to concrete structures, showing how additional protective measures can be combined to extend design service life. The measures considered are categorised into three types:

1. Basic material resistance type (e.g. water-cement ratio, cover, binder type, etc.).
2. Measures that increase the threshold for corrosion (e.g. corrosion inhibitors, etc.).
3. Measures that give the surface of the concrete added resistance to provide enhanced protection from aggressive agents in the service environment (e.g. by coatings, etc.).

Nolan *et al*. (2006) indicate that when combining measures to enhance service life it is most effective to choose complementary measures from different categories, rather than choosing measures of the same type.

The concept of 'criticality' provides a measure of the consequence of durability failure upon the functionality of the overall asset or a component part. The designer should therefore classify the component parts in the design into one of the following categories.

Consequences of durability failure			
Maintainable	2	1	1
Durability sensitive	**3**	2	1
Durability critical	**4**	**3**	2
	High	**Medium**	**Low**
	Degradation risk level posed by service environment		

Consequence score key:	
1	No additional measures required at concept design stage.
2	Additional protective measures may be adopted at concept design stage. Engineer's judgement.
3	Additional protective measures recommended at concept design stage. Engineer's judgement.
4	Additional protective measures should be considered at concept design stage.

Figure 5.15 Decision matrix for consequence of failure and durability risk (after BRE IP3/06 – Part 1: Nolan *et al.*, 2006). Courtesy of BRE

1. *Durability critical:* where failure of a component part would result in a significant and adverse affect upon the functionality of the asset, such that it is unable to provide the desired minimum level of performance. Durability failure therefore has the potential to cause major disruption and unacceptably high costs. These costs would typically comprise direct costs (i.e. to undertake the repair) and consequential costs (i.e. those due to disruption/loss of service). Durability critical component parts of an asset are therefore required to last its expected lifetime without an intervention being made.

   For example, the failure of a series of bridge beams may be durability critical where their failure causes unacceptable disruption to the functioning of a roadway underneath. Also if works to maintain a component part are expected to have an unacceptable impact on the functionality of the bridge, then the component part would need to be classified as durability critical. This would of course depend on the owner's requirements for continuity of the function of the bridge. Durability critical assets or component parts may require additional protection measures to reduce the risk of durability failure or to minimise the need for intervention within the required service life, thereby achieving a satisfactory through-life performance.
2. *Durability sensitive:* where failure of a component part would adversely affect upon the functionality of the asset, but not to such an extent that it is unable to provide the desired minimum level of performance. Although the efficiency of operation or the functionality of the asset might be reduced, maintenance, remedial or replacement works could be carried out for an acceptable cost and associated environmental impact.

   However, if the necessary remedial or other works were delayed excessively or not undertaken for some reason, it is conceivable that the functionality of the asset could be adversely affected in the longer term. For example, if the joint sealants in the cladding of a tall reinforced building were to experience deterioration it is unlikely that the structural functionality of the building would be immediately impaired. However, this situation might permit the ingress of moisture that would potentially cause corrosion and the risk of falling debris, which could pose a hazard to pedestrians passing the building. This is likely to be deemed to be an unacceptable hazard and would therefore potentially affect the functionality/use of the building and/or the zone around the building concerned.
3. *Maintainable:* where deterioration of a component part does not impact upon the performance of the asset such that the functionality of the asset is not adversely affected. Thus maintenance, remedial or replacement works can be carried out without undue inconvenience or disruption and for an acceptable cost and associated environmental impact.

In making this categorisation, all component parts should be considered carefully and the designer should be mindful of the consequences of a lack of performance or physical failure of seemingly unimportant items.

The designer should also consider the level of redundancy in the structure (e.g. simply supported structures are likely to have lower redundancy than a structure utilising continuous beams). The smaller the degree of redundancy in a structure the more critical durability failure tends to become. However, the designer should also be aware that unchecked deterioration of a component part of a structure with a high degree of redundancy, while possibly taking longer to fail than a similar component in a structure with a lower degree of redundancy, could eventually result in a more catastrophic and/or widespread failure.

Box 5.4 What is the required service life?

BS EN 1990 – *Basis of structural design* defines design working life (service life) as:

The assumed period for which a structure or part of it is to be used for its intended purpose with anticipated maintenance but without major repair being necessary.

BS EN1990 gives the following examples:

Design working life category	Indicative design working life (years)	Examples
1	10	Temporary structures (see Note 1)
2	10–25	Replaceable structural parts (e.g. gantry girders, bearings)
3	15–30	Agricultural and similar structures
4	50	Building structures and other common structures
5	100	Monumental building structures, bridges and other civil engineering structures

Note 1: Structures or parts of structures that can be dismantled with a view to being reused should not be considered as temporary.

Some owners have been known to specify service lives of perhaps 300 years or more on some forms of very long-life infrastructure. The actual end of use can be determined by a number of factors including changes of use and economics, as well as technical failure/non-compliance of the structure or its parts, as is discussed in the main text.

There are many factors in the design and construction process that can determine whether an asset will meet its design service life. These problems highlight the need for:

- A holistic approach to design embracing the entire construction process and explicitly addressing service life.
- Whole life costing in assessing the cost performance of constructed work and in deciding between alternative means of achieving the owner's objectives.

- Consideration of sustainability issues and whole life environmental impacts.

This approach may, at first sight, seem overly sophisticated for the majority of constructed assets, which are presently built to a simple prescriptive design. However, even a simple, straightforward, consideration of service life concepts at the design stage can radically improve the through-life performance of such an asset.

Service life design demands a sophisticated understanding of the loads and actions that can lead to premature failure and how those vary with time. We now have a good understanding of the mechanisms that can lead to deterioration and loss of serviceability, but our ability to reliably relate these to service life is still being developed.

There are three key steps to achieving the required service life:

- Identifying, as precisely as possible, what service life is required for the asset.
- Identifying, again as precisely as possible, the factors that could result in the structure failing to achieve its service life.
- Ensuring that at each stage of the construction process, during design and execution, as well as with respect to in- service maintenance and management activities, steps are taken to overcome these factors and their consequences.

Box 5.5 Idealised service life behaviour

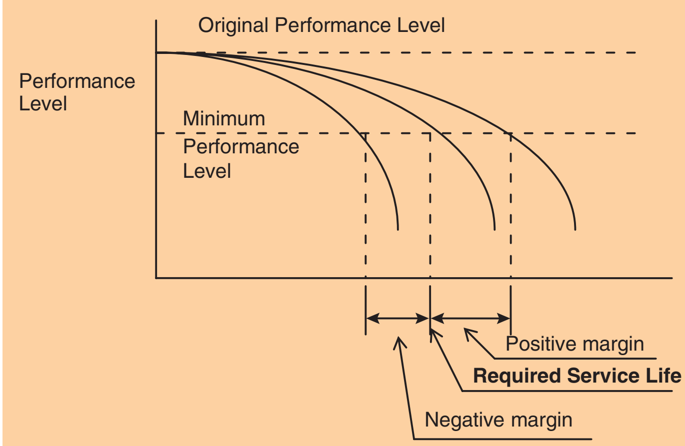
The following figure presents an 'idealised deterioration curve' which shows a simplified conceptual relationship between service life and structural performance level.

As is discussed in Box 5.4, the first step is to establish an explicit definition of what is meant by the 'required service life'. Consideration is then given to the mechanisms of deterioration acting upon the particular asset and also an appropriate model to represent through-life behaviour in these circumstances. These considerations can be applied at both the level of the structure and its component parts. Box 5.4 presents several examples of the required service life. However, these definitions are by no means all-encompassing or are set down in enough detail to establish tightly what condition will trigger the notional end of service life. There are a range of possible interpretations, which makes the potential outcome uncertain, and therefore not entirely satisfactory in terms of achieving clarity and management of expectations between the design team and the owner of the future asset. The matter of service life definition is also discussed in the main text.

The actual service life of the structure will not necessarily be equal to the originally required service life. The difference is known as the 'time margin', as shown in the figure below. If the actual service life is longer than the required service life, there will be a 'positive time margin'. Conversely, if the actual service life is shorter than the required service life, there will be a 'negative time margin'.

When designing it is necessary to have an adequate 'positive time margin' to be confident of achieving the required service life. This issue needs to be considered not only from the engineering perspective, but also from the viewpoints of the economic and other non-technical requirements. Based on the design scenario chosen, the owner will be able to establish more precisely the through-life management and maintenance strategy requirements.

It might be argued that a 'positive time margin' represents overdesign for the anticipated requirements, and that a 'negative time margin' represents underdesign and premature 'failure'.



Idealised deterioration curves for a constructed asset or part thereof

5.5 Service life design for durable constructed assets

5.5.1 Introduction

Critical aspects of planning and implementing through-life care for an asset that is to be constructed are the service life period for which the asset is designed and the associated through-life management and maintenance plan which is to be put in place to support this.

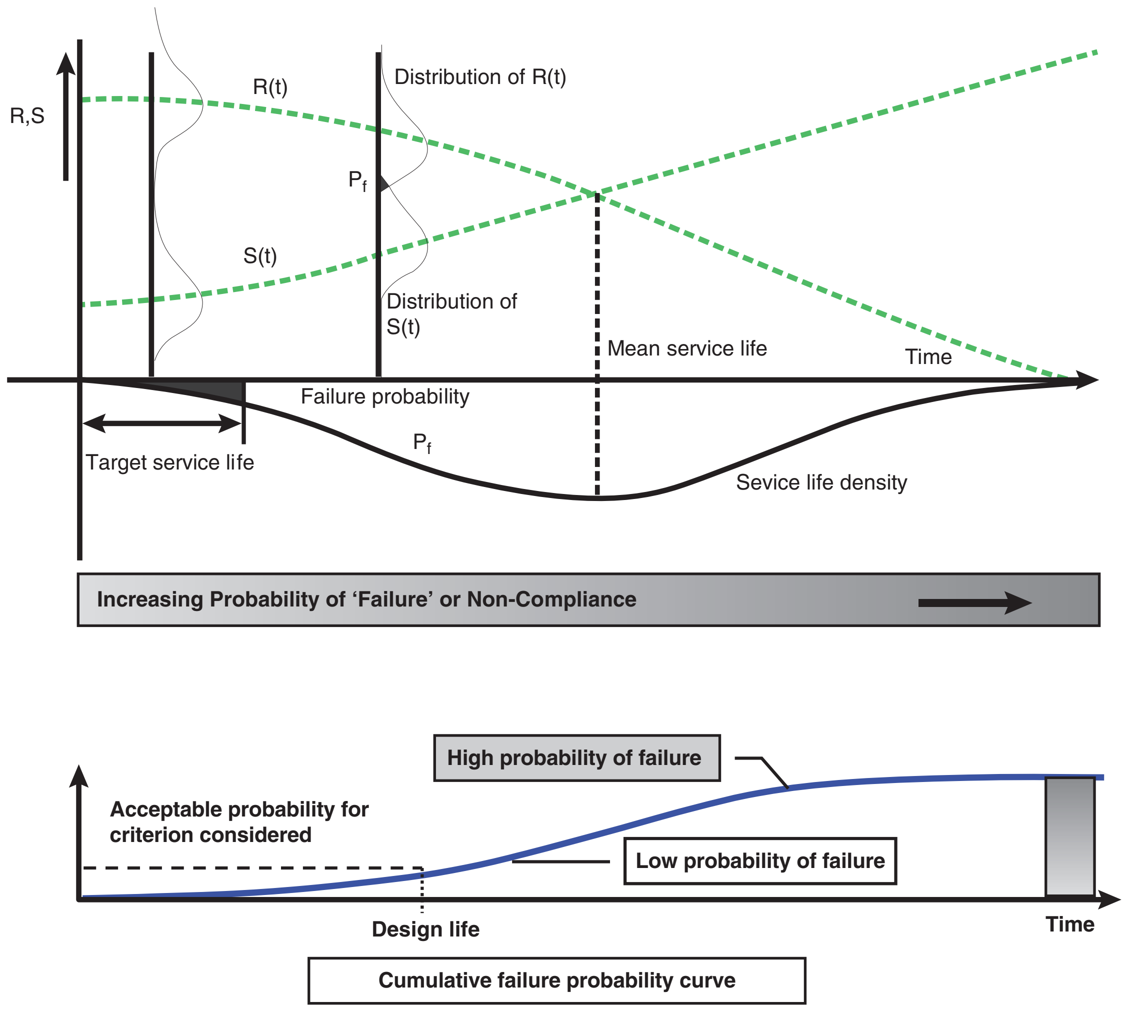
The design or required service life is the assumed period for which an asset, or part of it, is to be used for its intended purpose, with anticipated maintenance but without major repair being necessary. Design service life is defined by the selection of appropriate values for the following parameters:

- appropriate performance limit states;
- service life period (number of years);
- the level of reliability of not passing the relevant limit states during the chosen period.

Box 5.4 discusses what is termed the required service life, which is also variously referred to as the design service life and as the working life. Box 5.5 presents an idealised representation of service life behaviour, introducing the concept of a deterioration curve that might be employed to represent the through-life structural performance, or some other attribute, of a constructed asset. **Figure 5.16** presents a schematic illustrating graphically the concept of probabilistic service life design.

The durability of the asset in its environment needs to be such that it remains fit for use throughout its design service life. This requirement can be achieved in various ways:

- By designing suitable protective or mitigating systems.
- By using materials that, with appropriate maintenance, will not significantly degenerate during the design service life.
- By over-dimensioning the components of the structure that will experience deterioration to compensate for the prospective deterioration during the design service life (i.e. by sacrificial provision/over-sizing of the cross-section of components).



Where:

R(t) is some form of resistance function, shown here as time dependent. This might be concerned with structural performance (e.g. strength) or could be a durability related resistance (e.g. depth of cover in a concrete structure) - which would, of course, not be time variant/dependent.

S(t) is some form of loading function, shown here as time dependent. This might be a structural load (e.g. applied loading) or could be some form of environmental load or burden (e.g. depth of carbonation or chloride ion content/effects in the case of a concrete structure).

Note 1: In the case of structural performance, R(t) might be the ultimate load capacity (e.g. strength) of the member which, if affected by progressive deterioration, would be expected to decrease with time. For a standard structural loading regime, S(t) would not normally vary with time (e.g. the line would be horizontal). In situations where the imposed load was subject to revision over time (e.g. such as with highway loadings which historically have periodically been increased to take account of heavier vehicle weights), the line would have a stepped profile with time.

Note 2: In the case of the durability of a concrete structure, R(t) might be the depth of cover concrete. In the case of an environmental load the burden would generally be expected to increase with time; S(t) would increase with time (e.g. the line would slope upward with time, depending upon the type of environmental load involved, but the increase would not be linear with time).

Figure 5.16 Schematic illustrating the concept of probabilistic service life design

- By choosing a shorter lifetime for the structural elements and making provision for them to be replaced one or more times during the design life.

The above need to be undertaken in combination with appropriate inspection activities at fixed or condition dependent intervals, as well as in conjunction with appropriate maintenance activities.

It is interesting to note at this juncture that virtually all current structural design codes make no allowance for the effects of deterioration during the lifespan of the structure. This includes the structural Eurocodes developed by CEN, which are generally considered to be the most technically advanced suite of structural design codes currently available. In these it is assumed that the intended design performance level will be maintained by appropriate through-life management, care and maintenance activities. It should be recognised that, unfortunately, this is not always the case.

In 2010, the Fédération Internationale du Béton (*fib*) – otherwise known as the International Federation for Structural Concrete – published the final edited version of its *fib* Model Code 2010 as *fib* Bulletins 65 and 66 (*fib*, 2012). This is a full revision of the earlier CEB-FIP Model Code 1990 (CEB-FIP, 1992) which focused mainly on the structural aspects associated with the design of concrete structures. However, *fib* Model Code 2010 takes a holistic through-life perspective upon design, construction and in-service care and management of concrete structures. Accordingly *fib* Model Code 2010 also addresses issues concerned with matters such as sustainability, through-life care, impacts and management, as well as providing detailed and extensive guidance upon structural design issues. This broader treatment of the wide range of issues involved is reflected in a consistent manner in the recommendations made concerning execution, the selection and use of materials and conservation, as well as in the associated service life design concepts. It is thought that *fib* Model Code 2010 is the first structural design code to adopt this philosophical approach.

5.5.2 What is service life?

Box 5.4 discusses the issue of the required service life. However, in practice there are numerous different definitions of what constitutes the service life of a constructed asset, depending on the type of performance being considered. Five examples of other service life definitions are given below:

- *Technical service life:* the actual time in service until a defined minimum acceptable performance (functional) state is reached, perhaps due to deterioration associated with the service environment.
- *Functional service life:* the actual time in service until the structure, component or system becomes obsolete due to changes in the performance (functional) requirements, probably due to changed operational requirements.
- *Economic service life:* the actual time in service until replacement of the structure, component or system is economically more advantageous than bearing the maintenance/intervention costs associated with keeping them in service.
- *Extended service life:* the increased length of time the structure, component or system is required to remain in service due to changed performance (functional)/operational requirements; this is longer than the length of service life anticipated at the time of design and construction.
- *Achieved (or realised) service life:* the time that the structure, component or system actually remains in service (and presumably performing satisfactorily) until decommissioned, dismantled, demolished or otherwise removed from service.

It should be noted that generally the definitions of the above and related issues given in different guidance documents, standards and codes of practice are not entirely consistent.

5.5.3 Structural and service life design considerations

Structural and service life design needs to achieve appropriate levels of safety, in addition to seeking to ensure that the constructed asset is durable for the chosen design service life. These issues are addressed by establishing the required level of reliability for the structural and service life (durability) design and the associated requirements for design supervision and execution control.

Should a structure experience deterioration in service, consideration would need to be given to the potential implications of the nature and rate of deterioration upon the strength of a structure, as noted above. This would involve making a prognosis of the change in condition and strength with time, together with the length of time before any critical limit states (e.g. cracking or spalling of concrete, critical reduction in strength, etc.) will be reached. **Figure 5.16** presents a pictorial representation of this, showing how these circumstances would be addressed in the probabilistic service life design approach.

5.5.4 Approaches to service life design

Service life design approaches may be divided into two principal design strategies:

Service life design strategy A:	Avoiding deterioration by the design-out approach.
Service life design strategy B:	Providing resistance to the deterioration mechanisms active in the service environment.

The two principal service life design (SLD) strategies can be subdivided into a number of different methodologies for achieving the service life design objectives of the particular design strategy. The approaches adopted for the various structural materials differ in terms of their application and practicality in various circumstances.

It is perhaps easiest to illustrate the concepts in respect of one structural material, and this will be done for structural concrete. A classification of the various service life design strategies and associated methodologies for structural concrete is given below.

SLD Strategy A: Avoiding deterioration by the design-out approach on the basis of:

A1. Changing the service environment to remove the deterioration mechanisms.
A2. Using non-reactive materials to avoid potential deterioration reactions.
A3. Inhibiting the potential deterioration reactions.

SLD Strategy B: Providing resistance to the deterioration mechanisms active in the service environment on the basis of adopting:

B1. Deemed to satisfy (code) provisions.
B2. A single or multi-stage (barrier) protection strategy:

- **B2.1** Basic resistance using a single protection strategy.
- **B2.2** Enhanced resistance using a multi-stage protection strategy.

B3. The factorial method (adapted from BS ISO 15686: Part 8 (BSI, 2008)).
B4. A reliability-based methodology:

B4.1 Full probabilistic design.
B4.2 Deterministic partial-factor design (semi-probabilistic design).

The different methodologies outlined in respect of Strategy A – 'Avoiding deterioration by the design-out approach' – cannot be guaranteed to provide complete protection to structures in all circumstances. In most cases, they seek to minimise the risk of deterioration occurring under a range of envisaged circumstances. In some respects methodologies A1–A3 might be considered to be alternative protection schemes.

There is some overlap between the different methodologies providing resistance; for example, SLD strategy B3 potentially overlaps with activities in B1, B2 and B4. The service life design approaches assume that appropriate maintenance activities, such as clearing drains to ensure that water is shed from the structure quickly, will be undertaken in support of the chosen strategy.

In the context of concrete structures, the observations that may be made about the alternative approaches to service life design include:

- *Avoidance-of-deterioration method:* This is also known as the 'design-out' approach whereby potential deterioration mechanisms are avoided by removal of one or more components of the potential deterioration mechanism. Thus the problem of deterioration is overcome, or at least minimised as far as possible, by actions such as the selective use of stainless steel, the use of non-reactive aggregates, etc.
- *Deemed-to-satisfy method:* This approach involves the application of tabulated values for water–cement ratio, the depth of cover for the reinforcing bars, the reinforcement provision to control crack development and crack width, etc. This (recipe) approach is simple to apply and is commonly employed in codes of practice and other guidance documents (e.g. BS EN206 (BSI, 2000-6) for concrete structures).
- *Use of single- or multi-stage protection:* This methodology seeks to select appropriate measures to reduce the likelihood of deterioration, working either singularly or in combination to slow down the penetration of aggressive species into the concrete. These measures are commonly classified under the headings of materials, design and construction (Somerville, 1999). The main steps in the process are to:
  - Identify the nature of the deterioration mechanisms (hazards) present in the service environment, their aggressivity and the transport mechanisms responsible for creating the problems
  - Establish what parts of the structure may be at risk.
  - Select means of creating barriers to prevent or slow down the transport mechanisms.
- *Factorial approach:* There are two ways in which the factorial method described in BS ISO 15686: Part 8 (BSI, 2008) has been employed in service life design. These are (a) to make a *basic* estimate of service life, and (b) to make a *refined* estimate of service life.

  The latter is able to take account of measures that provide additional resistance or protection to the structure, component or system under consideration to extend its design service life. In BRE Information Paper 3/06 Part 3, Nolan *et al*. (2006) provide guidance on how the approach might be applied to a concrete structure.
- *Partial factor method:* This approach is based on design values for loads, capacities and geometrical characteristics and is the approach widely used for structural design.
- *Full probabilistic method:* This will seldom be possible for new structures due to lack of statistical data about environmental loads and material resistance functions or properties, but may be applicable to very important or major infrastructure projects or to the assessment /re-evaluation of existing structures where appropriate data can be obtained.

These approaches provide a framework within which strategies for the through-life management of the performance and care of structures can be developed. There are advantages and limitations associated with each of the approaches.

5.5.5 Example of the service life design process for a concrete structure

The main stages of a service life design procedure as applied to a concrete structure are illustrated in **Figure 5.17** which presents a simplified representation of the activities and steps involved. The actual process will be more complex with various steps being conducted in parallel and/or iteratively.

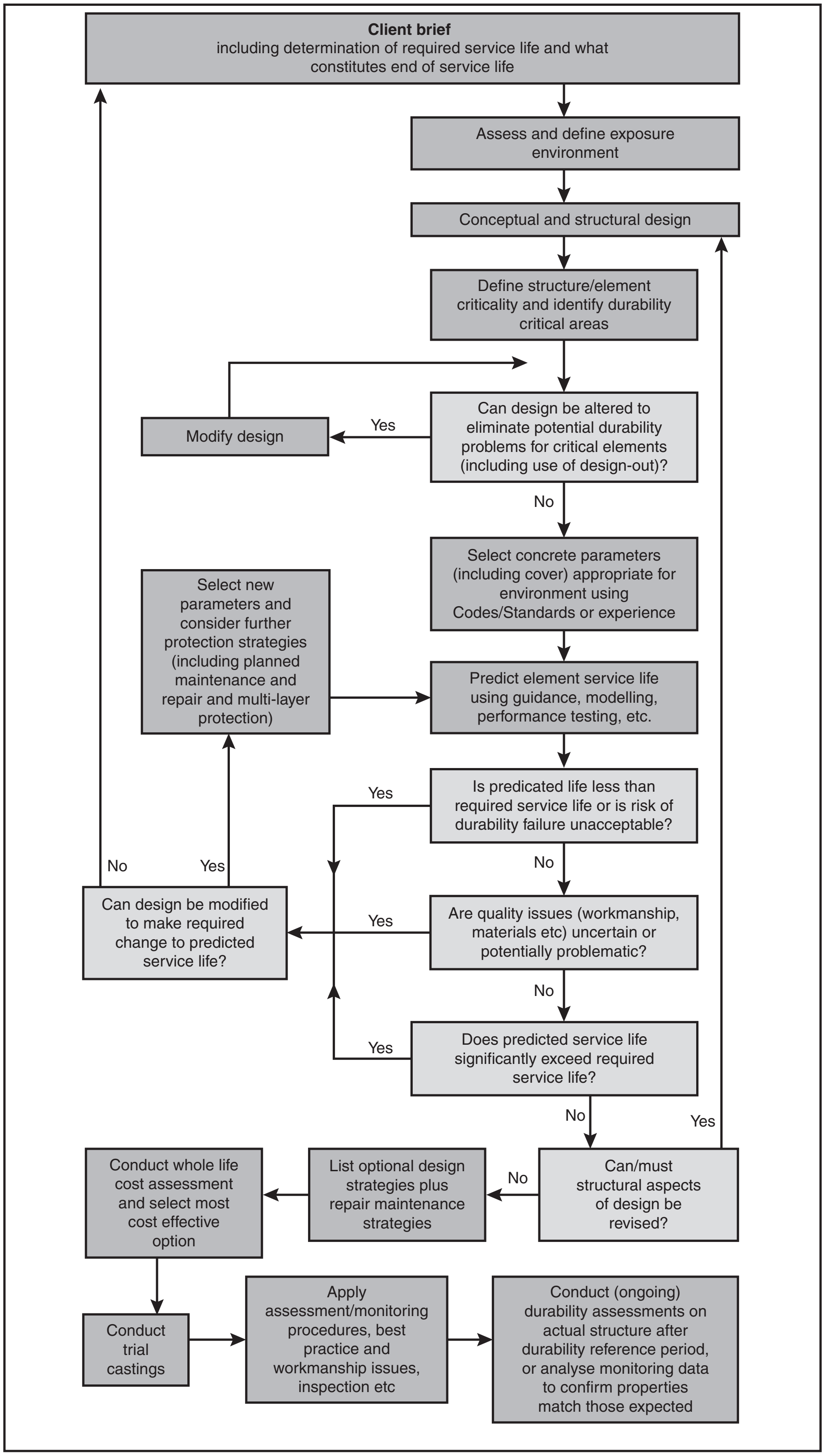
When the required target design service life has been fixed, the particular durability related design issues which need to be considered include the following:

- Structural layout and geometric form.
- Structural system.
- Location of expansion joints and construction joints.
- Environmental aggressivity at macro, meso and micro-climate scales.
- Critical deterioration mechanisms.
- Selection of materials and concrete mix.
- Concrete cover.
- Maximum design crack widths.
- Special protective measures required.
- Execution methods.

When taking a through-life perspective, life-cycle costing is an important consideration. The service life design process can be used as a basis for a life-cycle costing assessment. Using this process, technically equivalent options (i.e. those with a high probability of achieving the owner's requirements) can be compared and evaluated over the life of the structure. Notionally a similar approach might be adopted in respect of environmental impacts.

Service life design process: The main procedural steps associated with service life design option selection typically involve:

1. Specifying the required (target) design service life.
2. Defining the exposure environment – see Section 5.6.3.



Note: The flowchart shown here is simplified. The service life design procedure is likely to be iterative, or with steps conducted in parallel

Figure 5.17 The main steps in a service life design procedure for a concrete structure (Quillin, 2001). Courtesy of BRE

3. Establishing the minimum requirements defined in standards or codes.
4. Choosing preliminary concrete mix design/protection measures.
5. Making preliminary predictions of the anticipated service life on the basis of the defined environmental conditions and various concrete mix or protection combinations.
6. Comparing the predicted service life with that required.
7. Making any design modifications required and defining any additional protection measures required to achieve the target service life.
8. Establishing a 'shortlist' of additional measures options, if this is appropriate, considering cost and environmental impact implications.
9. Selecting a suitable combination of measures for the required (target) design service life.

5.6 The structural and service life design, construction and through-life care processes

5.6.1 Introduction

Service life design should be an integral part of the overall structural design process, rather than a stand-alone 'bolt-on' carried out subsequently as part of a quality assurance process. However expressed, some form of required service life is essential to enable alternative design options for meeting it to be evaluated at the conceptual and detailed design stages.

Figure 5.18 presents a greatly simplified representation of the main stages in a service life design approach, and its broad interactions with structural design. The general flow in the process is from the top of the diagram to the bottom, but it also portrays lateral interactions between the structural and durability design processes. Most important is the feedback loop that seeks to recognise the implications of certain decisions

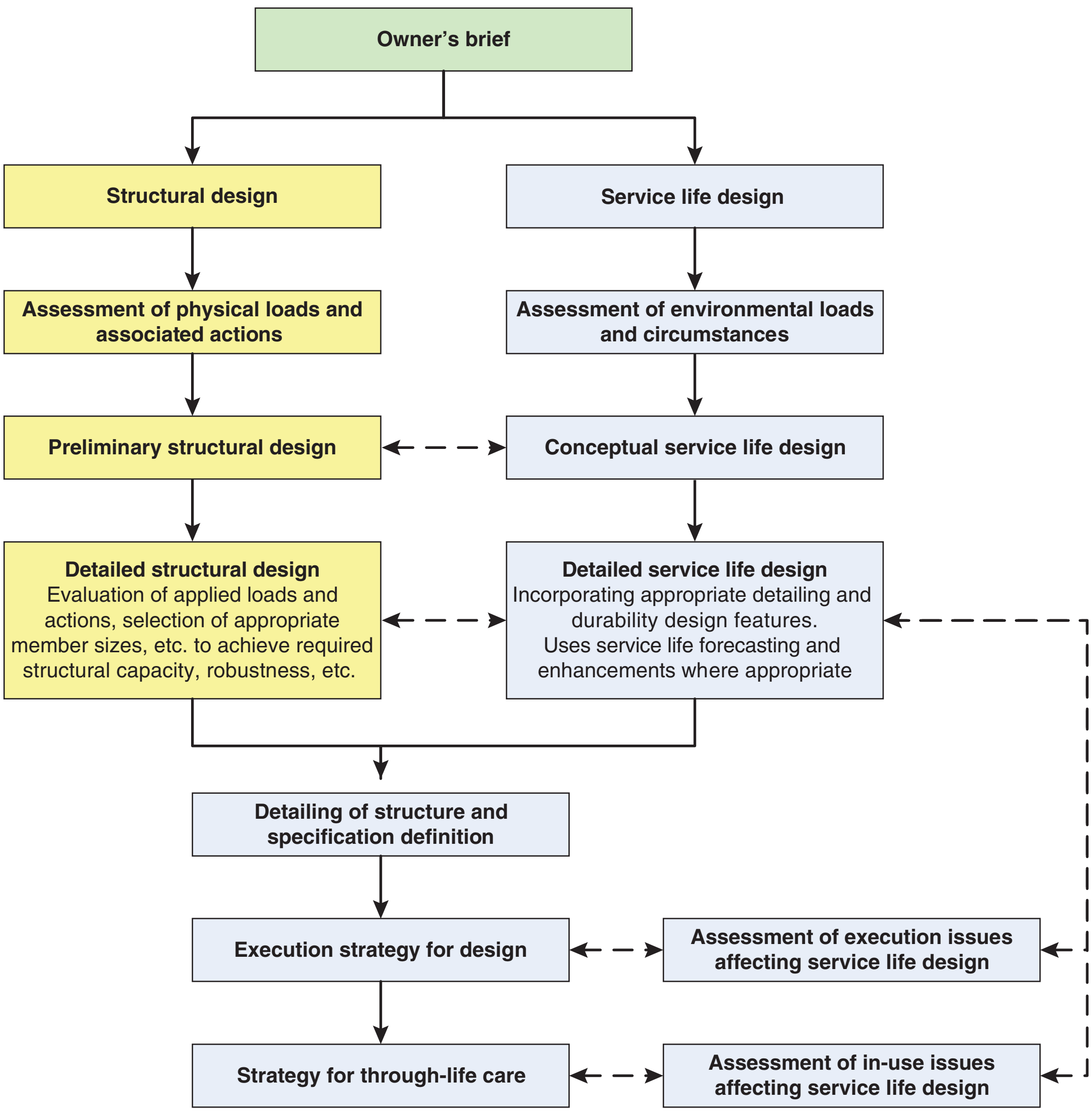


Figure 5.18 Simplified service life design procedure (Nolan *et al.*, 2006)

associated with the way the structure is to be constructed (the execution strategy) and/or maintained. For example, if it is envisaged that minimal maintenance will be carried out, the service life design process will need to introduce appropriate measures to ensure that adequate durability is achieved.

The system utilises well-established concepts and current best design practices that involve the following main process steps:

- *The owner brief:* defining owner requirements and expectations.
- *Assessment of environmental loads:* characterising the exposure environment.
- *Conceptual design:* creating a concept meeting the owner's requirements.
- *Detailed design:* designing and detailing for durability and buildability.
- *Execution strategy:* designing for buildability and on-site supervision levels.
- *Maintenance strategy:* ensuring continuity of functionality of the asset and providing the user with the required information.

In addition, there is also the possibility of taking account of experience gained from the performance of existing structures in the current design process, recognising the facets of design and detailing those that facilitate greater longevity and durability. In many instances, these relate to the effective shedding of water from the structure, component or system concerned.

In short, the structure needs to be designed so that it is constructible (buildable), accessible, inspectable, maintainable and repairable.

In recognition of the current focus on all aspects of environmental impact, these matters need to also include consideration of making elements replaceable where appropriate. End-of-life issues include the adaptation of the structures concerned to new applications, dismantling and the reuse of structural elements where possible, together with demolition and the recycling of the associated materials, where direct reuse of the elements is not practical. It is anticipated that there will be an increasing future focus on environmental impact issues.

The following are essential aspects of the overall process:

- Adopting a holistic approach, ensuring that service life and durability are considered throughout the structural design process. For example, considering the way the structure interacts with its environment and how maintenance will affect operation and its service life.
- The concept of estimating the likely service life based on the design, its environment and other factors, with the objective of ensuring that the service life achieved exceeds the target length of the design life.
- Enhancing the design or specification in the light of specific advice based on state of the art knowledge of materials to meet the design service life.

All the following parties should contribute to the service life design process in different ways. They all have important contributions to make.

- The owner, by defining clear performance requirements and taking an appropriate life-cycle (long-term) perspective upon decisions, costs and benefits to be accrued.
- The designer and other technical professionals, by converting the owner's performance requirements into design concepts, strategies, technical specifications and conditions.
- The contractor and material suppliers, by using appropriate materials and processes in a manner that follows the technical specifications and design concepts in order to ensure that the structure will meet the owner's performance requirements and will be adequately durable.
- The operator (if not the owner), by providing through-life care and management, undertaking timely maintenance and interventions upon the structure to ensure that the intended service life is achieved without unforeseen failures or unnecessary costs being incurred.

Any of these parties may, by their action or inaction, contribute to a lack of durability and excessive future costs for remedial works upon the structure, with the possible disruption costs to users associated with any loss of service or functionality that this may involve.

However, first and foremost, the owner:

- should demand good quality construction;
- must check the quality received (or have it checked by others);
- must be prepared to pay an adequate sum to get the required quality (i.e. the initial construction cost);
- must maintain the structure in a satisfactory condition and be prepared to bear the costs of this.

The philosophy behind such an approach was more elegantly described as 'The Common Law of Business' attributed to John Ruskin (1860):

> *It's unwise to pay too much, but it's worse to pay too little.*
>
> *When you pay too much, you lose a little money – that is all.*
>
> *When you pay too little, you sometimes lose everything, because the thing you bought was incapable of doing the thing it was bought to do.*
>
> *The common law of business prohibits paying a little and getting a lot – it can't be done.*
>
> *If you deal with the lowest bidder, it is well to add something for the risk you run.*
>
> *And if you do that, you will have enough to pay for something better.*

Related sentiments in a contemporary context were voiced more recently by Lord Rea in the House of Lords in January 2003 in an observation upon the importance and value of achieving a high quality in design:

> *Good design may initially cost a little more in time and thought, although not necessarily in money. But the end result is more pleasing to the eye and more efficient, costs less to maintain and is kinder to the environment.*

The value of achieving good quality in design, the issues associated with this and how the achieved quality can be assessed are discussed in Box 5.3. Design Quality Indicators (DQIs) assess design quality under the three main headings of *impact*, *build quality and functionality* (see also **Figure 5.7**). DQIs address a wide range of issues that go well beyond consideration of the functional structure of the building or constructed asset, with the critical success factors for achieving design quality being described in Box 5.3.

More information about DQIs can be found through the Construction Industry Council website at www.dqi.org.uk. Other specialist tools have been developed for specific sectors; these include the *Achieving Excellence Design Evaluation Toolkit* (AEDET) for the UK National Health Service (NHS) and the *Design Excellence Evaluation Process* (DEEP) for UK Defence Estates.

5.6.2 Owner brief and performance expectations

The owner brief addresses the relevant needs and aims of the construction project, resources to be provided by the owner, the details of the project and any appropriate design requirements within which all subsequent briefing (when needed) and designing can take place. This stage of the design process should highlight the important issues for the owner (and parties advising and assisting the owner in the preparation of the brief) to address in order to enable an effective service life design to be undertaken. It may be subdivided into two parts; the owner's basic needs and the consequent performance requirements, as described below.

Owners vary greatly in their understanding of how to formulate and express their needs and a key role of the designer/consultant is helping the owner to understand and express these needs in terms that can be translated into clear performance requirements.

Owner's basic needs: The owner should consider a number of basic issues that can influence the design and the performance requirements for the facility (i.e. the proposed building or asset that is to be constructed). These include:

- What type of facility is needed and where is it to be located.
- The planned function(s) of the facility and its component parts.
- The performance requirements for the facility and its component parts.
- Any desired sustainability credentials.
- Appearance or aesthetic requirements, both initially and through life.
- Requirements for usable space, dimensions, services and fittings.
- The period of tenure and the requirements for the facility at the end of this period.
- Future changes of use to increase flexibility and minimise the risk of obsolescence.
- Restrictions to the design (e.g. planning regulations).

Definition of the owner's basic requirements will enable the performance characteristics for the structure to be specified. These *performance requirements*, together with an assessment of the environmental loads, will form the basis of the design solution and should include:

- Safety and serviceability requirements.
- Service life and clarity upon what constitutes the end of service life.
- Importance of continuity of function and flexibility to accommodate changes of use.
- Management and maintenance requirements.
- Acceptable level of inspection, maintenance, repair or replacement.
- Acceptable life-cycle or whole-life costs (capital and operating costs).
- Design restrictions.
- Agreed sustainability credentials and targets for minimising environmental impact.

The issue of life-cycle or whole-life cost encapsulates an understanding of how design decisions affect not only the initial construction costs, but the costs of operating the structure throughout its life. It is expected to become an increasingly significant influence in the design process, along with sustainability considerations. However, as with service life design, there is still much work to be done to translate ongoing research in these areas into a usable system.

Most of what follows is concerned with durability related aspects of service life design; however, other considerations may apply in particular circumstances (e.g. fatigue, thermal performance effects, etc.).

5.6.3 Environment aggressivity classification

The environment to which the structure will be exposed is a key factor in designing for a given service life. Relevant factors include:

- The general environment conditions of the area in which the structure is situated (macro-climate).
- The specific location and orientation of the surface being considered and its exposure to prevailing winds, rainfall, etc. (meso-climate).
- Localised conditions (micro-climate) such as surface ponding, exposure to surface run-off and spray, aggressive agents, regular wetting, condensation, etc. These aspects include those arising as a consequence of the interaction between the structure and the environment (e.g. cladding to keep the structure dry or ponding due to poor detailing, etc.). It is essential that the interaction between the structure and its environment is addressed when seeking to design durable structures. The influence of an aggressive environment on potentially vulnerable and critical components can be reduced by appropriate design. These issues would be considered in the 'conceptual design' and 'detailed design' phases of the service life design process.
- The required performance under the defined environmental loads.

Environmental loads are defined as aggressive actions, natural or man-made, transmitted to the structure through ground, water or the atmosphere, which, directly or indirectly, can attack structural components, through chemical or physical means, potentially reducing their load-bearing capacity, stability or functional serviceability and appearance.

In the service life design process the assessment of the environmental loads is brought together with outputs from the owner's brief to provide a definition of the required performance under these loads. This forms the starting point for the durability-related aspects of the conceptual design stage in which the basic defence strategy is formulated.

Figure 5.19 illustrates some possible micro-climate conditions for a bridge subjected to de-icing salts. Water can be present on the structure as a result of ponding, rundown and spray effects. Water may also be in the form of vapour and condensation, as well as from leaks and seepage through joints and cracks. The effect that water from different sources has upon the durability of the structure will vary, principally depending upon how they contribute to the different transport mechanisms facilitating the penetration of de-icing salts or other aggressive species into the structure. Bridge orientation and the direction of the prevailing wind will also influence micro-climate conditions on the bridge. However, many of the potential problems can be alleviated by detailing, particularly in terms of sloping surfaces to avoid ponding, by making provision for movement and by removing water from the structure as quickly as possible.

5.6.4 Example of an environmental aggressivity classification system for concrete structures

As an example the following presents the system of environmental aggressivity classification for concrete structures given in BS EN206–1 (BSI, 2000). The approach recognises the principal forms of deterioration that affect concrete structures and defines these in terms of the aggressivity of different service environment conditions. The standard specifically differentiates between exposures leading to reinforcement corrosion and exposures leading to deterioration of the concrete.

The various exposure classes for concrete structures defined in BS EN206–1 (BSI, 2000) are as follows:

XO: No risk of corrosion or attack

XC: Corrosion of reinforcement induced by carbonation

XD: Corrosion of reinforcement induced by chlorides other than from seawater

XS: Corrosion of reinforcement induced by chlorides from seawater

XF: Freeze–thaw attack upon concrete

XA: Chemical attack upon concrete

However, in the UK it is deemed appropriate to consider a wider range of environmental actions than is covered by the base European Standard. For example, the UK guidance on this matter is given in BS 8500: Parts 1 and 2 (BSI, 2006), which uses the approach given in BRE Special Digest 1: *Concrete in*

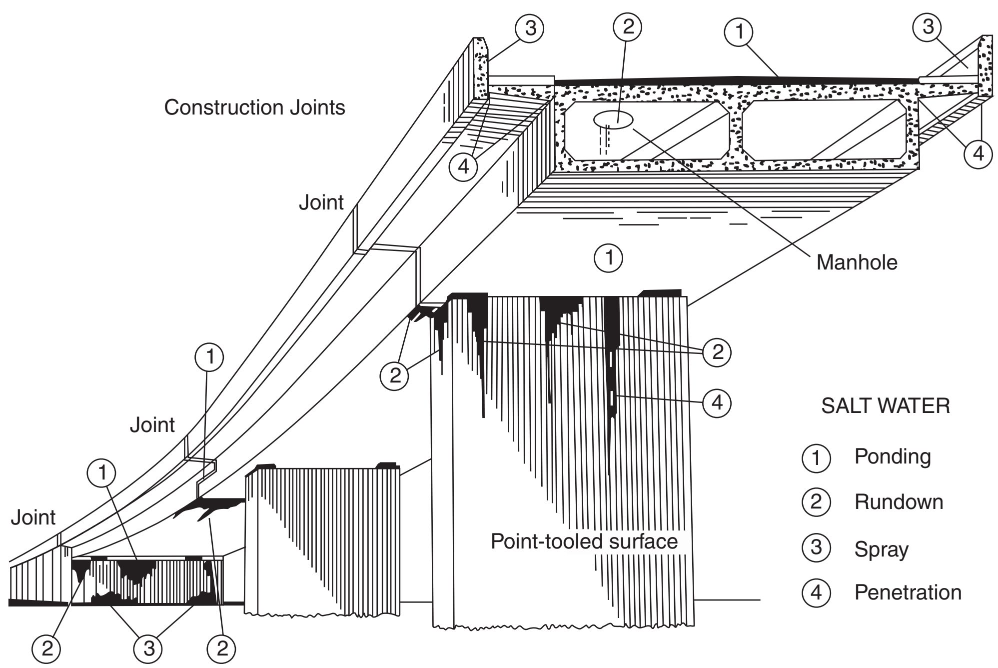


Figure 5.19 Some micro-climate conditions on a bridge subject to de-icing salts. Reproduced from Pritchard (1992), courtesy of Bruce Pritchard

Aggressive Ground (BRE SD1, 2005). This adopts a system for the classification of the aggressivity of ground conditions to concrete known as the *Aggressive Chemical Environment for Concrete Class* (ACEC Class) which takes account of a number of factors including the soluble sulfate and magnesium, the potential sulfate (e.g. from oxidation of pyrite), the type of site (i.e. natural or brownfield) and the mobility and pH of the groundwater.

5.6.5 Conceptual design for durability

Conceptual design forms the core of the service life design process. Based on an assessment of loads and of the owner's requirements key decisions are made regarding how best to resist the environment and the choice of structural form to minimise the risk of future deterioration, whilst meeting structural performance requirements. The aim should be to minimise the effects, which are influenced by wind and temperature, of water by ensuring that water drains off the structure quickly and by reducing penetration in durability-critical areas. For routine structures this may be straightforward, involving some combination of appropriate prescriptions with good detailing practices. In more extreme situations greater rigour may be needed, perhaps using a design-out approach or a multi-layer protection system – or possibly a combination of the two concepts with appropriate detailing practices, utilising either the factorial (Nolan *et al*., 2006; *fib* Bulletin 53 (2010)) or reliability-based methodologies (*fib* Bulletin 34 (2006), *fib* Bulletin 53 (2010)) to evaluate alternative options.

At this stage decisions need to be made to maximise the sustainability of the structure, based on the sustainability credentials agreed in the performance requirements. In particular, issues such as designing for reuse, minimising waste and energy in use need to be addressed at this point.

The likely level of workmanship and the quality of materials need to be taken into account. Decisions made at the conceptual design stage will also influence the maintenance and management strategies. Parts of the building or constructed asset that may be vulnerable to deterioration need to be identified, along with their criticality, and whether it is possible for them to be maintained or whether they may need to be replaced during the design service life. Life-cycle or whole-life cost assessments of potential design options may be carried out prior to selection of the most appropriate option. These assessments may require initial forecasts of component service lives in their specific exposure environment. These assessments may be carried out in more depth at the detailed design stage.

The ease with which a design can be translated into reality (i.e. its 'buildability') needs to be assessed throughout the design process. Structures and details that are difficult to build may be constructed to a lower quality than those that are easier to build, with potentially significant consequences for durability. It is advisable that the construction team are able to comment on the buildability of the design. It is also important for the designer to be aware of the construction methods that may be preferred by the prospective contractors for the project and to know their advantages and constraints.

The selection of the basic defence strategy during conceptual design is one of the key decisions affecting durability. The two main service life design strategies that can be used have been outlined in Section 5.5.4:

Service life design strategy A:	Avoiding deterioration by the design-out approach.
Service life design strategy B:	Providing resistance to the deterioration mechanisms active in the service environment.

Depending on the circumstances, there are a number of options (methodologies) within each which can be employed to achieve the design objectives.

5.6.6 Detailed design for durability

In this stage, the conceptual service life design is developed alongside the conventional structural design process into something that can be built. Conventional structural design is essentially a numerical process involving structural analysis and section (element) design undertaken to satisfy the specified limit states, but also there are the additional and very important facets of dimensioning and detailing of the structure.

For example, in the context of a concrete structure, the following items would need to be included in the output from structural design process, as these are critical to the durability of the structure:

- Appropriate concrete cover (also relevant to bond and fire resistance considerations).
- The required concrete strength grade and concrete composition.
- Reinforcement arrangements to limit flexural cracking under dead or imposed loads, particularly crack widths, allow compaction of concrete, etc.
- Shapes, dimensions, jointing of components and general detailing; to provide adequate stiffness and stability, to limit deflections, control water flow and exposure to aggressive species, but also to ensure ease of construction and buildability, etc.
- Consideration of movement, under the effects of temperature, wind, etc.

Any areas that are critical to durability and that require particular attention need to be identified and addressed during the detailed design stage. These should be as easy to build as possible. Minor elements with short service lives need to be easy to replace. Inspection, maintenance, repair and replacement plans should also be developed at this stage. These plans help to ensure that the performance of the structure matches that required by the owner.

5.6.7 Codes and standards – Deemed-to-satisfy durability design

The 'deemed-to-satisfy' provisions are typically given in tabulated form within national and international codes of practice for structural design and/or the associated materials standards. For example, in the case of concrete structures, these generally relate the provision of resistance (e.g. cement type and quantity, maximum water–cement ratio, the depth of cover, concrete grade, minimum air content, type of curing, control of early age cracking, limitation of crack widths, etc.) to the aggressivity of the service environment and the length of design service life.

The deemed-to-satisfy provisions are generally accepted as being the minimum requirements or provision that designers are expected to meet. The provisions may not reflect current best practice measures because of the time it takes to achieve consensus and to go through the relevant standardisation procedures associated with the periodic updating of such guidance. However, the deemed-to-satisfy provisions in the more modern and international codes of practice for structural design do generally result in structures that are durable in most service environments. However, these provisions may not be adequate in the most severe or aggressive environments. Furthermore, recent studies (e.g. Matthews, 2010) indicate that the levels of reliability achieved by various deemed-to-satisfy durability provisions are not consistent (i.e. they vary significantly between the different provisions adopted in various circumstances) for different environmental conditions and circumstances.

The deemed-to-satisfy provisions within national and international codes of practice for structural design are expected to remain the principal basis for the durability design of most structures in 'normal' service environments for the foreseeable future.

Typically national codes and standards for structural design do not yet include service life design approaches using a reliability-based (probabilistic) methodology which are becoming a more realistic possibility for structures with long or very long service life and/or are exposed to severe/aggressive environments.

Deemed-to-satisfy provisions do not provide a means of predicting the service life of a structure based upon data gathered about a particular service environment. For the same reason, they do not permit service life updating where a revised estimate of the service life of the structure is made based upon data gathered from in-service measurements on the structure.

5.6.8 Probabilistic performance-based service life design

As noted previously, there are parallels between contemporary structural and probabilistic performance-based service life design. The widely used prescriptive approach to durability design has limitations in that it does not recognise the inherent variability in the environmental 'loading' applied to the structure and also the potential 'resistance' of the materials used to form the structure. This variability is similar to that dealt with in the structural design process, where variations occur in both the physical loading/actions applied to the structure and in the mechanical properties of the materials. For example, for ultimate limit state design BS EN1990: *Basis of structural design* (BSI, 2002) describes various combinations of actions to be considered in the design process to represent the variability physical loading/actions that might occur in practice, namely for the:

- persistent design situation;
- transient design situation;
- accidental design situation;
- seismic design situation.

For structural design it is usual to assume that the strength (resistance) remains constant, and that the loads, even if fluctuating, can be characterised by a single value. Thus the (load and resistance) variables are assumed not to vary with time. Partial safety factors are applied to both the load and response functions, to take account of the variability and uncertainties, and to produce values that can be used for design.

Probabilistic performance-based service life design takes the concepts employed in contemporary structural design and extends them to the durability design of structures. As durability involves time-dependent parameters, the simplifying assumptions used in structural design cannot be employed in durability design. Accordingly the procedures employed in probabilistic performance-based service life design are somewhat more complicated than those employed for contemporary structural design. In addition, there may be several serviceability related limit states associated with the end of the technical service life to consider when designing for durability (see Section 5.5.2). **Figure 5.16** illustrates the concepts of probabilistic service life design.

5.6.9 Other considerations influencing service life design

Designers have traditionally recognised that the acceptability of the design and the performance of a structure or asset will be judged against a range of criteria such as structural safety, serviceability and first (construction) cost. In recent years designers have increasingly come to appreciate that the suitability of a design solution needs to be verified against a greater number of criteria including sustainability and whole-life or life-cycle cost.

The *fib* Model Code 2010 – published as *fib* Bulletins 65 and 66 (*fib*, 2012) – has recognised this situation and defines various performance criteria against which a design solution should be verified. These include structural safety, serviceability and sustainability limit states, recognising that the whole-life or life-cycle cost of the proposed solution should be considered rather than just its first (construction) cost.

In this regard, structural safety considerations may include issues such as stress limits, strength capacity limits, fatigue, structural stability, seismic response and safety, as well as robustness and progressive collapse limits, etc.

Serviceability limit states relate to fitness-for-use considerations that affect the functionality of the constructed asset in its normal use, as well as other aspects such as the comfort of using the asset. The associated performance requirements may include criteria such as deformation limits, crack width limits, vibration limits, etc.

Sustainability considerations can involve a diverse range of issues such as:

- Environmental aspects – e.g. toxicity impacts, material usage relative to resource availability, pollution issues with respect to discharge limits for emission to air, water and land, land use and ecology, energy usage, water usage, waste produced, etc.
- Societal aspects – involving evaluation of the impact of the asset upon society during construction and whilst in operation, management aspects, health and well-being, transport links, consideration of aesthetics, etc.

In addition, value management and value engineering review procedures will often have a bearing upon the nature of the final scheme taken forward for construction.

5.6.10 The project specification

The durability of a structure and its associated through-life costs and environmental impacts are largely determined by the achievement of appropriately specified durability related requirements (or conversely the failure to define and/or to achieve them) set down in the project specification for the design and construction of an asset. The project specification, incorporating the execution specification and project-specific quality plan requirements, provides the owner and the owner's professional team with a means of defining and achieving the standard of performance they require.

It is critical that appropriate information is included in the tender and contract documents which provide the legal and financial foundation for procuring the desired asset. For example, it is essential that the owner defines the length of service life required for the building or constructed asset. This should be clearly stated in the project specification.

Owners and their professional teams need to know how to draw up appropriate tender and contract documents to ensure that problems do not occur that could diminish the actual durability of the structure below the level that is required. The centrepiece of these documents is the project specification. The goal is that these documents should incorporate the necessary preventive action requirements and effective quality control procedures and that they should not simply result from a 'form for form's sake' based approach to quality management.

These issues relate particularly to the execution of construction and to quality plan requirements when the structure is being built.

The term 'execution' refers to all the physical activities undertaken for the physical completion (construction) of the works. For example, for a concrete structure these activities might include procurement, scaffolding, falsework, formwork, reinforcement, concreting, curing, etc., as well as the related inspection and documentation of those activities. This usage of this term is consistent with the convention adopted in European and international technical construction standards.

5.6.11 Execution of works

The execution phase is crucial in determining whether the structure will be durable for the design service life. There have been numerous examples of durability problems due to poor construction practices, but these have often originated from design issues. For example, these deficiencies can lead to corrosion induced damage, reductions in structural capacity, as well as creating the need for repair works and a larger than anticipated maintenance burden.

The quality of workmanship is influenced by the quality of design and detailing. Some designs are difficult to build, even with reasonable care and standards of workmanship on site. Many construction problems could be avoided by involving the contractor in the design process, in accordance with modern concept of partnering.

From an environmental perspective it is important to note that there is considerable wastage of materials on site, unused during the construction process. In the case of UK construction, it is reported (McGrath and Anderson, 2000) that historically some 10–15% of all materials delivered to site were wasted. Much of this waste is avoidable and has been estimated to reduce the already small profits of construction companies by perhaps as much as 25%. One such approach to minimising waste is summarised in Box 5.6.

Box 5.6 Auditing and minimising construction waste on site

There are various site-based methodologies to audit waste streams generated during the construction/execution process, with the goal of targeting reductions in these. These methodologies are intended to provide robust and accurate mechanisms by which wastes arising can be benchmarked and categorised by source, type, amount, cause and cost. Audits are typically undertaken for activities such as construction, demolition, refurbishment, manufacturing and prefabrication. The data provide a springboard to identifying and prioritising actions to reduce waste (which is a producer responsibility), to achieving reuse of waste streams at source (the proximity principle), and maximising recovery to extend the effective life-cycle of all types of construction materials. The software tools available often enable the potential cost savings to be identified and the reduction, reuse, recycling and recovery options of materials to be maximised. One such system is SMARTWaste™ (Site Methodology to Audit, Reduce and Target Waste) which was developed by the Building Research Establishment (BRE).

5.6.12 Through-life care and maintenance considerations

If the structure is to meet the defined performance requirements it is necessary to ensure that the structure is being used in a way that is compatible with the design intent and that inspection, maintenance, repair and replacement are conducted in accordance with the plans developed under the detailed design stage. End of life decisions, such as the implications of obsolescence, possible changes of use and the loss of fitness for purpose must also be addressed. These issues influence the design and so must also be considered as part of the owner brief and conceptual design.

5.6.13 Birth and Re-birth certificate documentation

The concepts of 'Birth certificate' and 'Re-birth certificate' can be applied to structures and various forms of constructed assets. The following definitions for these concepts are adapted from those given in *fib* Bulletin 44 (*fib*, 2008) for concrete structures:

> *Birth certificate*: A document, report or technical file (depending on the size and complexity of the structure concerned) containing engineering information formally defining the form and the condition of the structure after construction. The document/report should provide specific details on parameters important to the durability and service life of the structure concerned (e.g. for a concrete structure – cover to reinforcement, concrete permeability, environmental conditions, quality of workmanship achieved, etc.) and the basis upon which future knowledge of through-life performance should be recorded. This framework should provide a means of comparing actual behaviour/performance with that anticipated at the time of design of the structure. The document/report should facilitate ongoing (through-life) evaluation of the service life which is likely to be achieved by the structure.
>
> *Re-birth certificate*: Similar to the 'Birth certificate' for a structure, but relates to the information and circumstances associated with a project for the repair/remediation/refurbishment of the structure or a part thereof to extend its anticipated service life.

Thus a 'Birth certificate' could act as a basic element of a predictive life-cycle management system for an asset. Adoption of this concept would provide improved knowledge of current and future condition states of a structure, which could give a basis for the assessment of future structural resistance for known or anticipated environmental loadings by the use of appropriate deterioration models. The evaluations would be undertaken as required in combination with (a) material testing to establish relevant material properties during the compliance stage verification and (b) non-destructive test methods during acceptance testing after completion of the structure. The approach would allow optimisation of structural resistance during design, the review/improvement of the condition prognosis for the structure based upon Bayesian updating using material and performance data gathered from the final structure (the completed works), as well as the use of focused preventive or remedial interventions where this was necessary.

5.7 Concluding remarks – Future challenges and opportunities

5.7.1 Introduction

There are many developments that may potentially have an influence in the coming years upon durability issues, service life design and matters related to the through-life performance and care of constructed assets. Whilst it is not possible to give a comprehensive overview of these, the topics include items such as:

- Moves towards a low carbon future.
- The wider adoption of building information modelling (BIM).
- Influence of climate change – a future need to adapt buildings.
- Improved scientific understanding of deterioration processes.
- Developments in materials.
- Life-cycle cost optimisation to reduce overall through-life cost.
- Life-cycle environmental impacts and sustainability considerations.
- Developments in codes and standards for design and execution of constructed assets.
- Process changes, such as more emphasis on factory production of building elements.

Perhaps the most important of these potential influences are the first three listed; these are discussed in more detail below to illustrate some aspects of their potential impacts. A number of the other topics have been considered previously in the document text.

5.7.2 Moves towards a low carbon future

Along with others, the construction industry is subject to a range of influential drivers pushing it and the UK and world economy towards a low carbon future.

As a result the construction industry faces profound challenges to meet the desired improvement in the standards of new building in response to climate change and related issues. UK examples of these drivers of change include evolving regulatory requirements, such as the UK Climate Change Act which has enshrined in law a commitment that the UK will achieve a 26% reduction in overall CO_2 emissions by 2020 and an 80% reduction by 2050. These are challenging targets. Associated drivers introduced by the UK government which are influencing the UK construction industry include:

- The introduction of the Code for Sustainable Homes (DCLG, 2008).
- An ongoing requirement to reduce energy usage and measures to raise awareness of energy usage, e.g. the current evolution in Part L of the UK Building Regulations, a well as the introduction of Energy Performance Certificates (EPCs) and Display Energy Certificates (DECs).

Future objectives include the construction of 'zero' operational carbon residential buildings after 2016 and for other new buildings from 2019, which involve considerable future implications for both new and existing buildings and the associated building envelopes. It is envisaged that there will be an important future role for the designer/structural engineer in achieving a satisfactory and potentially innovative balance between embodied and operational 'carbon'. Furthermore, as operational 'carbon' reduces to low levels in the future, the relative importance of embodied carbon will increase. Thus in the future, designers will be in the forefront of the quest to find appropriate design and engineering solutions that combine low embodied carbon with adequate durability for the potential long intended service life of the assets concerned. This is expected to be a challenging task

If carbon trading is introduced more widely in the future, it is expected to become a significant influence upon the selection of design solutions.

5.7.3 The adoption of building information modelling (BIM)

Building information modelling (BIM) has many different facets and potential uses. At its heart is a digital model of the building or constructed asset which brings together all the relevant information associated with the design, construction and through-life management of such assets. The information can be readily shared, updated and interrogated by all those concerned with the project, with all the relevant data being held in one 'data environment'.

BIM enables attributes to be assigned to elements of the asset defining their characteristics and their relationship to other elements, as well as the ability to bring together multiple data sources to provide a comprehensive representation of the asset in question. In principle, BIM can be used for all stages of a construction project, from cradle to grave or cradle to cradle. Potentially this might involve all through-life activities from compliance checking, 'clash' identification, design, environmental impact and/or embodied carbon analysis to construction planning, procurement management, facilities management post-construction, decommissioning, dismantling or demolition. In essence, BIM is a single-source coordinated information set. Accordingly it is more than just the geometric information contained in a three-dimensional computer-aided design (3D CAD) model.

The UK government has indicated that it intends to adopt BIM for the procurement and management of public assets. It sees BIM as a means of improving the effectiveness of these processes and reducing costs. Reports in the technical press suggest that currently only about one-third of the UK construction industry has adopted it. Other countries are embracing this technology. Since 2007, Denmark and Finland have required BIM to be used on all public sector construction projects. Take-up of BIM is reportedly higher in the USA than in the UK and Western Europe; for example, the US Coast Guard and the General Services Administration both require BIM to be used for certain functions. In the USA, most BIM adoption has occurred since 2007.

The adoption of BIM is expected to drive significant change in the UK construction industry. The UK government initiative proposes a phased take-up of BIM over a five-year period starting in 2011.

5.7.4 Influence of climate change – a future need to adapt buildings

Whilst the UK construction industry is already working to make buildings more energy efficient, the expected changes in the climate over the next 100 years mean that there will be a need to adapt our existing and new buildings to cope with the changed conditions. For example, it has been suggested that the UK could be faced with:

- Wetter winters and drier summers in every part of the UK, with potential implications for surface water run-off, groundwater conditions and foundation performance.
- A decrease in average summer rainfall in the south-east of the UK of about 20%; the south-east is an area that is already water-stressed and already needs to reduce water usage.
- An increase in average winter rainfall in the north-west of the UK of about 16%, with increases in the amount of rain on the wettest days leading to a higher risk of flooding.
- A rise in sea level by up to 36 cm, affecting coastal areas and tidal zones of rivers.

As a result significant changes are planned for standards and regulations for buildings. Accordingly there will be a need for the construction industry to develop approaches for the adaptation of buildings to respond to these forthcoming changes. These issues will affect both proposed low impact buildings and existing buildings that are to be refurbished to low impact standards.

Owners and their professional teams will need to consider a range of issues associated with the adaptation of buildings to meet the potential requirements of future climate change. Potential questions include:

- What is the current risk exposure for the building to projected future climate change?
- What is the best way to adapt specific buildings, now and in the future, to improve resilience to climate change and thus extend their commercial viability?
- On what basis will decisions be made about implementing adaptation measures?
- What is the best way to undertake adaptation work?

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CONTENTS

Chapter 6

Controlling the design process

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In order to reduce the incidence of faults in structural design, careful attention to the processes used is essential. In particular, faults in the use of structural analysis and of structural codes of practice can lead to serious consequences. The basic strategy is to adopt a questioning approach to all inputs and outputs with special attention to validation of models and verification of results of calculations.

6.1 Introduction

The consequences of a structural failure can be very serious. Large numbers of people can be killed in a single incident. The most serious structural failure in the UK in terms of loss of life was the 1879 Tay Rail Bridge collapse causing the deaths of 75 people – mainly due to a modelling error (Martin and MacLeod, 1995). Although there is now much better control over structural design and construction processes, failure is still a risk that is uppermost in the minds of structural designers.

One of the main issues in relation to risk is that computer implementation of calculations is becoming dominant. This has led to the view that modern structural designers are losing contact with the calculations leading to poorer understanding. That such a situation exists may be true but it is not a necessary consequence of computer use. Understanding calculations is less related to how they are performed than to how they are approached. This chapter describes an approach to doing calculations that helps to reduce the risk in their use and helps to develop understanding.

The fundamental principle being promoted is that there are basic strategies that can be applied to all processes in order to help to reduce faults in their use. These strategies are discussed mainly in relation to using structural codes of practice and using structural analysis models. Such process control is mainly about adopting a questioning attitude to all inputs and outputs.

6.2 Basic control strategies

6.2.1 Basic principles

The following definitions are used here (Institution of Structural Engineers):

- *Validation*: Consideration of whether a process is capable of satisfying the requirements.
- *Verification*: Consideration of whether the process has been correctly implemented.

Verification is commonly used but validation (in the sense stated above) is less commonly addressed in a formal way. It needs more attention than it normally receives.

Basic issues in relation to a process are:

- The requirements.
- The process itself.
- The outcomes.

The corresponding control strategies are:

- Assess the requirements – Have all the needed requirements been identified?
- Validate the process – Is the process capable of satisfying the requirements?
- Verify the results – Has the process been correctly implemented?

This is summarised in **Table 6.1**.

Review should be treated as a set of activities which are pervasive in the design process. Their use implies constant vigilance in the identification of faults.

6.2.2 Other control strategies

- *Process optimisation* – Just getting a process that is valid, i.e. that satisfies the requirements, may not be adequate. One should seek to identify the best possible process, i.e. the process needs to be optimised.
- *Results interpretation* – Results need to be checked but they also should be scrutinised for information about the behaviour of the system being designed.
- *Equip* – Assessing the need for competence, hardware, software, information, etc.

Activity	Outcome	Control	
Define the requirements	Requirements statement	Assess the requirements	Review
Define the process	Description of the process	Validate the process	
Implement the process	Process outcomes	Verify the outcomes	

Table 6.1 Basic process components and controls

Using the control strategies can be thought of as a formal approach – as a Quality Assessment process. It can also be viewed as a pervasive mode of thinking – of asking the right questions to seek to ensure that what is being done results in good outcomes.

6.3 The design process – Inception

6.3.1 Design requirements

A successful outcome in design is dependent on the adequacy of the requirements. With errors and omissions in the requirements the chance of a satisfactory outcome may be low.

Types of requirement for the design process include:

- Client service brief – what service the designer agrees to provide for the client.
- Client design brief – what the client wishes to be achieved from the product to be designed.
- Design requirements generated by the design team.

Best practice is to take the client brief and develop it into a comprehensive set of design requirements irrespective of the degree of detail in the client brief.

Some principles for developing design requirements are:

- Requirements must encompass objectives and constraints.
- The requirements should, where possible, be expressed in performance terms. Statements that will limit the form of the facility being designed should be avoided unless (a) there is no realistic alternative or (b) it is a client requirement.
- It is important to seek to establish the requirements at the earliest possible time. Changes to the requirements during the design phase will push up the design costs. Changes to the requirements at the construction phase can be very expensive.

The holistic view of design infers that all issues that may affect the outcome should be taken into account. To achieve this it is clearly necessary to have a focus on requirements (**Figure 6.1**).

6.3.2 Design information

Figure 6.2 illustrates the role of information in the design process. The input to, and the output from design are both in the form of information. If the input information is not optimum the success of the design outcomes is compromised.

Two types of information can be identified:

- Project information – client brief, site plans, surveys, geotechnical reports and the results of other investigations commissioned for the project.
- Design support in the form of codes of practice, cases of similar design contexts, textbooks, journal papers, guidance documents, etc.

In non-standard situations more resources may need to be allocated to creating project information and searching for design support information.

6.4 The design process – Concept design

6.4.1 Development of options

It is said that the choice of concept controls the quality and the cost of the final design. With an inadequate concept there is little chance of success. Careful attention to the choice of concept is an important feature of best practice structural design.

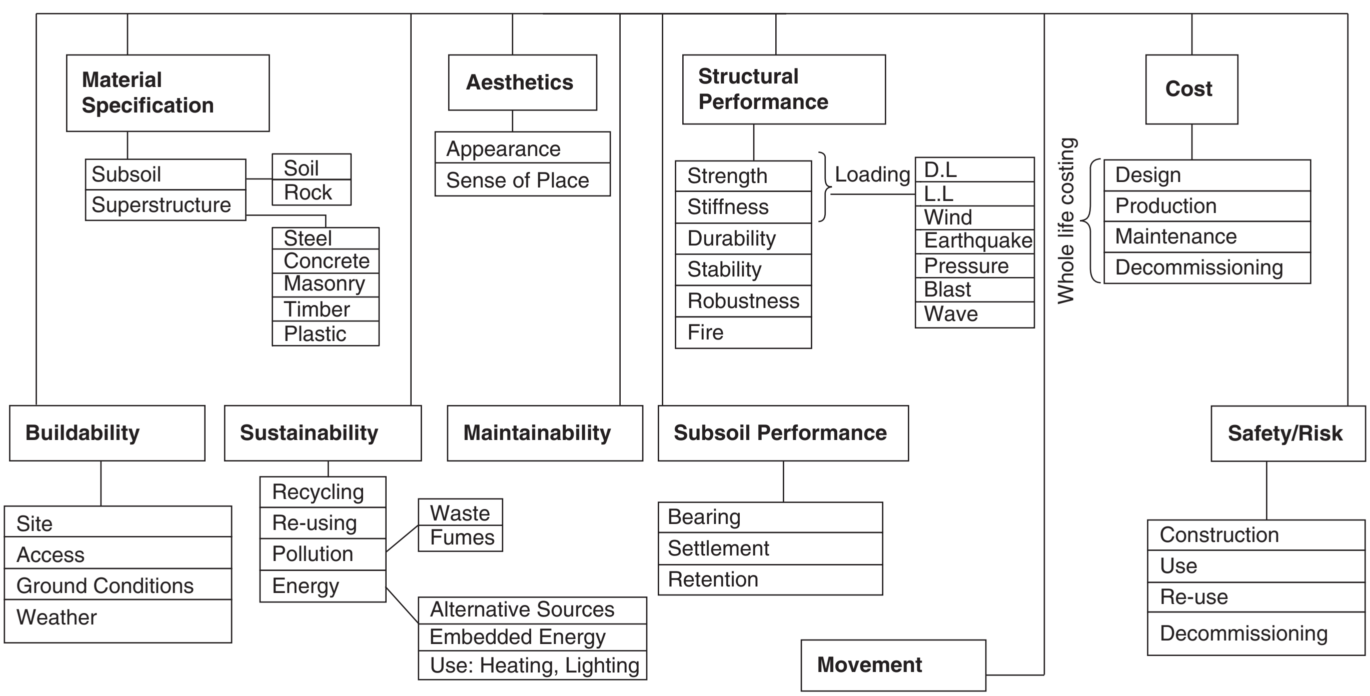

Figure 6.1 Generic issues in relation to structural design requirements

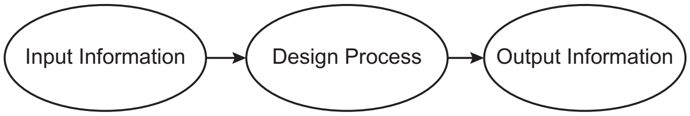

Figure 6.2 Information and the design process

Example 1 Four-storey building

A four-storey residential building was constructed with a steel frame, brick cladding and brick internal partitions. Would it have been cheaper to make the brickwork load-bearing and omit the steel? An advantage of having a steel frame is that longer floor spans can allow later adaptation of the building for other occupancy. In this case the steel columns were quite closely spaced so that there was not much scope for reconfiguring the layout.

Issues to be considered include:

- Cost of preparing the concept designs. In order to compare options, partial designs need to be developed. A balance between the cost of doing such work and its value in relation to the option assessment needs to be made.
- For a building, the relationship between the cost of the structure and the cost of the building. Getting the cheapest structure may not always result in lowest building cost.

Example 2 Five-storey building

For a five-storey building it was found that the savings in cladding by using a floor layout with low structural depth more than offset the extra cost of that type of floor.

6.4.2 Assessing options

An option can be considered to be a design concept which is capable of satisfying the requirements, i.e. a valid solution. A range of techniques is used to assess options – sometimes called 'multi-criteria assessment'. These tend to rank or score the options based on mostly subjective assessments of relative value for each of the assessment criteria.

Choosing the option to be used may be the most important decision in the design process. Care needs to be taken when comparing rankings of options based on criteria assessment. Most of the criteria have to be judged on a qualitative basis and there is likely to be a significant amount of uncertainty in making rankings/scorings on the basis of such criteria. An exception to this is cost, which can be defined quantitatively although there is likely still to be uncertainty about the values. If cost is of prime importance (it normally is) then it may be best to rank the options using the qualitative criteria and then compare these against cost. A healthy degree of scepticism about any ranking of options is advisable.

Note that just because the best option from a set has been identified, this may not be the optimum design solution. There may be other potential options that would be better.

6.5 Technical design

'Structural design' should be treated as an overarching concept and therefore traditional 'structural design' is referred to here as *technical design* meaning the use of codes of practice and other rules to establish adequate sizes for structural members.

Example 3 The London Millennium Footbridge 2000

At the opening of the London Millennium Bridge on 10 June 2000 (**Figure 6.3**) a wind was blowing over the deck. People crossing the bridge experienced an oscillatory sway which in some cases made balance difficult. The bridge had to be closed down until the cause of the problem was investigated and remedial measures taken.

The source of the vibration proved to be synchronous movement of the people on the bridge at the lateral first mode natural frequency of vibration of the bridge (Dallard *et al.*, 2001). In order to balance, the people on the bridge shifted their weight in time with this frequency. This phenomenon is known as synchronous lateral excitation. It had been experienced previously but information about it had not been well disseminated nor had the effect been quantified. It is quantified now. It can occur on bridges even of standard construction with lateral frequencies less than 1.3 hertz and with large crowds on the deck. The basic error was in the requirements for the technical design. Synchronous lateral excitation should have been considered.

Example 4 The Ronan Point collapse 1968

The 22-storey Ronan Point building in London was of 'large panel' construction, i.e. the structure of the walls and floors consisted of large precast concrete units with no beams and columns. In 1968, a gas explosion at the eighteenth storey resulted in the local walls being blown out. The fall of the now unsupported panels directly above the explosion then took away the corner panels below the explosion (Griffiths *et al.*, 1968). Four people died. The historic view of this event is normally that of a wake-up call to consider progressive collapse in structural design. But the inquiry into that disaster, and other investigations that followed, highlighted the equally important issue that many designers at that time did not perform well in innovative situations. UK designers of large panel buildings in the 1960s used the then current code CP114 Design of Reinforced Concrete Buildings. Little thought was given to the validation question, 'Does this code apply to large panel buildings?' Issues neglected included: tying of the panels together and provision for shear transfer at the vertical joints between wall panels.

6.5.1 Controlling structural design calculations

The control strategies as applied to the technical design of members and components can be formulated as follows:

6.5.1.1 Requirements assessment

The requirements can be interpreted as issues to be considered. For example in the design of a concrete beam one needs to take account of bending, shear, short-term deflection, long-term deflection, cracking, etc. At the outset one needs to identify the issues that need to be addressed. The designers of the London Millennium Footbridge can be excused for not taking account

Figure 6.3 The London Millennium Bridge

of the then unquantified synchronous lateral excitation but the designers of the Ronan Point building were held liable for the collapse because they failed to consider that the panels should be tied together.

6.5.1.2 Process validation

To address the design issues 'provisions' are used. Normally the provisions come from a national or international code of practice. Where suitable provisions are not available in the national code, the designer has to look to other sources, e.g. codes from other countries, design guidance, books and published papers. Sometimes there is no guidance and the provisions have to be developed ad hoc. This was the situation for tie action in the design of large panel buildings in the 1960s. In some cases, the need for tie action was not recognised (i.e. the requirements were deficient). In other cases, the need for tie action may have been recognised but was not adequately provided for.

From the provisions, a schedule of calculations needs to be drawn up and implemented.

The main validation questions are: Do the provisions adequately address the issues? Are they relevant to the context?

Four types of issue can be identified:

1. Codified – where provisions are available in a code of practice.
2. Not codified but quantified – at least one method of dealing with the issue has been published but it is not available in a code of practice.
3. Identified but not quantified – some people know about the issue but there are no published provisions for it.
4. Not previously identified.

Clearly there can be no justification for omitting a codified issue. Type 2 issues soon get incorporated into a code – this is how codes develop as new issues are identified. Designers may not be held negligent for omitting a Type 3 issue (the London

Millennium Bridge problem was of this type, as was the reason for the Tacoma Narrows Bridge collapse in 1938). Type 4 issues are rare.

6.5.1.3 Results verification

There are two parts to a results verification:

1. One has to ensure that the calculations correctly implement the provisions.
2. One has to ensure that the data used in the calculations are correct.

In the case of software implementation the first part involves answering the question: 'Has adequate resource been applied to ensuring that the coding for the software is error free?' The verification question for the second part is 'Has adequate resource been applied to checking the data for the run(s) of the software?' For hand calculations the question is: 'Have the calculations been adequately checked?'

6.5.1.4 Final review

An overall assessment of the results is a very important control feature. Questions to be asked include: 'Do the results look right?' 'How well do the results compare with those from previous similar projects?'

6.5.2 Hand calculations vs. computer calculations

It is normally asserted that doing hand calculations helps designers to understand better the basis of what they are doing. In fact, the potential for understanding when using code provisions tends to be low whatever strategy is used for doing the calculations. The provisions are normally stated in a form that makes it easier to do the calculations than to understand the reasoning behind them (see Example 5), and in many cases information is not available about the background to provisions.

Example 5 Case-study: Minimum reinforcement of concrete beams

In Eurocode 2 (for concrete structures) a minimum area of tensile reinforcement for beams is specified (Clause 9.2.1.1) which is directly related to the concrete strength and to the area of the concreted and inversely to the steel strength. It is difficult to understand why this should be so unless you know that the rule is based on the relationship:

$$M_y \geq M_{ct}$$

where M_y is the yield moment for the cracked section and M_{ct} is the moment at tensile cracking of the concrete (ignoring the steel). This makes sense because if the criterion is not satisfied, when the section cracks, a surge of load can be taken by the reinforcement which can snap rather than show ductile behaviour.

Computer calculations have the important advantages that the risk of arithmetical errors is reduced and calculations can be efficiently repeated. If it is ensured that (a) all the relevant issues are being addressed, (b) the provisions used are relevant to the context and (c) the provisions have been correctly implemented, then the risk in using computer calculations is probably lower than by hand calculations.

6.5.3 Programming calculations

Two basic strategies in computer programming are:

- *Procedural* – where the programmer decides how the relationships are connected together and what options are made available. This is implemented using programming languages such as BASIC, C, etc.
- *Declarative* – where the relationships or *rules* are given in a list and the system works out how they are to be processed. If the rules are numerical the software can be described as a *numerical rule processor* of which there are two basic types: (a) spreadsheets and (b) maths processors such as TEDDS and Mathcad.

For engineering calculations there are significant advantages in the declarative approach. Spreadsheets are good for handling tabular data but the maths processor approach is favoured for engineering calculations because (a) the likelihood of errors can be significantly less than with spreadsheets and (b) the layout of the calculations is similar to that of hand calculations

6.5.4 Doing calculations in standard and non-standard contexts

A *standard context* can be defined as a situation where typically:

- the designer has recently done a very similar design and is familiar with the details of a suitable process used, or
- the design is entirely within the scope of a code of practice – see **Figure 6.4(a)**.

In a standard context the amount of resource allocated to requirements assessment, to validation and to software verification may be quite small. But there is always a risk of implementation errors and results verification should always be treated as an important issue.

If a well-established program is available for a standard context, then there is less need for deep understanding of the context.

In a non-standard situation where the design context is outside the zone of familiarity (**Figure 6.4(b)**) it is very important to seek understanding of the behaviour of the system being designed and of the relationship between the system and the provisions being used.

6.5.5 Summary of design review questions for technical design

Questions to be asked include:

- Have all the design issues been identified?
- Is the design context within the scope of the rules to be used?
- Is the software capable of processing the rules? (software validation).

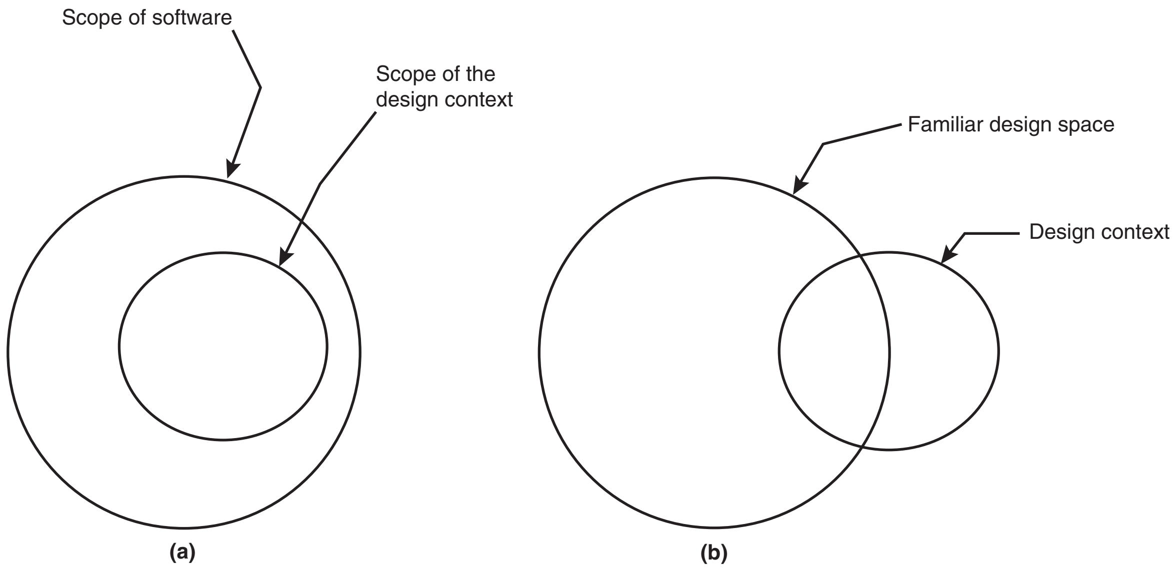

Figure 6.4 (a) Standard and (b) non-standard contexts

- Has the software been adequately checked for errors? (software verification)
- Are the data for the software correct?
- For hand calculations – are there errors in doing the calculations?
- Do the results look right?
- What quantitative checks can I do?

6.6 Analysis modelling

6.6.1 Background

A new view of structural analysis is emerging as no longer a calculation-dominated activity but as mainly a modelling activity (MacLeod, 2005). Most structural analysis calculations are implemented on computers and the conventional wisdom is that this leads to a decline in understanding structural behaviour leading to an increased risk of design errors. Such an outcome is by no means inevitable. By using the modelling process as set out in Institution of Structural Engineers (2002) and MacLeod (2005), understanding can be enhanced and the risk of faults can decrease.

That there is significant risk from improper use of analysis models is very evident – see Examples 6 and 7.

Example 6 The collapse of the Sleipner offshore platform 1991

In 1991 the construction of this massive concrete oil drilling platform was almost complete in a Norwegian fjord when it sank (Foeroyvik, 1991). The use of a finite element mesh of volume elements proved inadequate to predict the shear in a concrete wall. A simple 'back-of an-envelope' calculation could have given more accurate results for this particular parameter and could have prevented the total loss of the platform.

Example 7 The collapse of the Ramsgate Walkway 1994

This is an example of where a rudimentary error in the modelling of the distribution of force in a system caused a collapse (Chapman, 1998). The walkway was supported on stub axles cantilevered from the walkway frame and sitting on bearings. The designers assumed that the resultant transverse force on the axle would be near the fixed end of the axle and therefore the moment in the axle would be low (**Figure 6.5**). The position of the resultant was not close to the support and the axle failed in bending.

The *engineering model* is a representation of the structure that is to be analysed. The *analysis model* is a mathematical representation of the engineering model for the purpose of predicting behaviour under load, etc.

The analysis model normally involves two levels of assumption:

- For the *conceptual model* assumptions are made about the material behaviour, loading, support conditions, etc.
- For the *computational model* further assumptions are made to allow the conceptual model to be solved. In frame analysis the computational issues are less prominent than with models that are treated as plates of solids.

6.6.2 The analysis modelling process

The activities in the modelling process as advised in the IStructE *Use of Computers* report (2002) are: Plan, Equip, Model, Calculate. **Table 6.2** sets out the activities, outcomes and controls for analysis modelling using **Table 6.1** as a template.

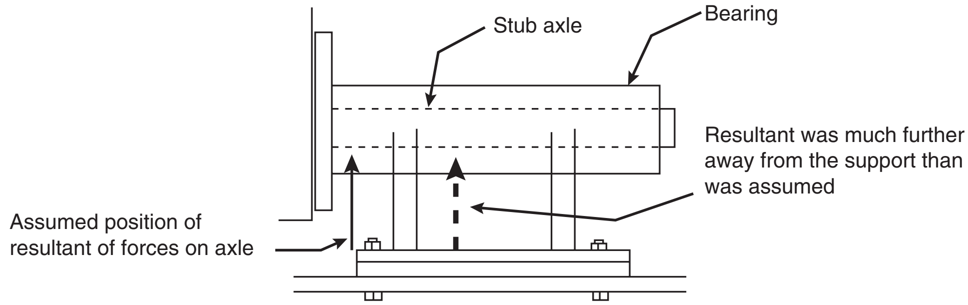

Figure 6.5 Stub axle bearing for Ramsgate Walkway showing positions of resultant force

Activity	Outcome	Control	
Define the requirements	Requirements statement	Assess the requirements	
Define the model	Description of the model	Validate the model	Review
Do the calculations	Results	Verify the results	

Table 6.2 Basic process components and controls for analysis modelling

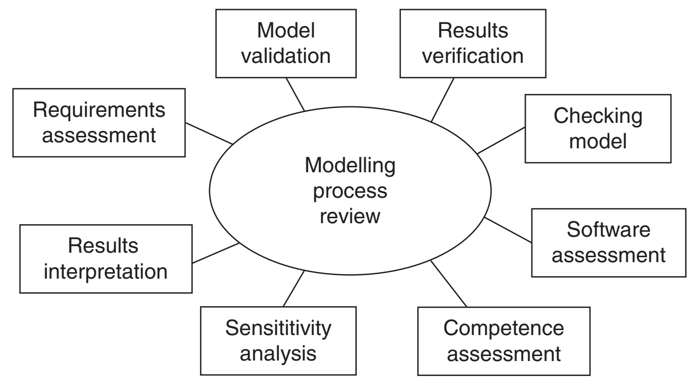

Figure 6.6 Activities in a modelling process review

6.6.3 Analysis modelling review

6.6.3.1 Modelling review activities

A modelling review is the sum of all the activities that seek to identify faults in the modelling process. **Figure 6.6** shows a range of modelling review activities.

6.6.3.2 Requirements assessment

While the requirements for a structural analysis are normally evident it is worthwhile to reflect on them and to draw up a requirements statement for complex or non-standard contexts.

Questions for requirements assessment include:

1. *Stresses and internal forces*: Have the locations and target accuracy been identified? Has the distinction between local and resultant stresses been clearly identified?
2. *Displacements*: Where do the displacements need to be defined and to what accuracy?
3. *Natural frequencies and mode shapes*: What range of natural frequencies need to be considered?
4. *Fundamental requirements assessment question*: Have all required performance issues been identified and included in the requirements statement?

6.6.3.3 Model validation

Model validation is the process of ascertaining whether the model is capable of meeting the requirements.

A model validation is carried out by listing all the assumptions made for the model. To do this one needs *validation information*, i.e. information which discusses the applicability of assumptions. Such information is not readily available in conventional texts on structural analysis – but see MacLeod (2005).

It is good practice to prepare a validation analysis for non-standard contexts. Ways of presenting these are given in Institution of Structural Engineers (2002) and MacLeod (2005).

While having an initial validation exercise is very important, validation information can emerge from the results especially from sensitivity analysis. A constant lookout for information that will assist the validation should be maintained.

Questions for validation include:

1. *Assumptions*: Have all the assumptions for the model been identified?
2. *Validation information*: Is all relevant validation information available?
3. *Test results*: Are there test results available to support the validation?
4. *Fundamental validation question*: Is the model capable of satisfying the requirements?

6.6.3.4 Results verification

Results verification is the process of seeking to ensure that the model has been correctly implemented.

Both formal and informal strategies should be used to seek to identify faults in the implementation of the model. A formal checklist can be used but also regular quantitative and qualitative checks should be carried out.

Verification checks include:

1. *Data checking*: Has there been sufficient resource applied to data checking?
2. *Overall equilibrium*: Has a check been made on overall equilibrium? For example, the sum of the total vertical reactions should be checked against the total vertical load that was expected to be applied to the model.
3. *Symmetry*: If there is symmetry has it been checked via a symmetric loadcase?
4. *Form of results*: Does a qualitative assessment of the results show any anomalies?
5. *Values of results:* Are the values of the results in the expected range?
6. *Checking models*: see Section 6.6.3.5.
7. *The fundamental verification question*: Has adequate resource been allocated to minimise the risk of implementation errors in the model?

6.6.3.5 Checking model

Checking the model against another frame of reference – a checking model – can be a valuable review activity. The checking model may take the form of:

- A 'back of an envelope' calculation, i.e. a hand calculation based on a simplified model of the system to provide a quick check.
- A simplified model of the system which requires software for the solution.
- A repeat of the model using different software and/or different personnel.

Simplified models should be assessed for validity. It is important that they are able to adequately represent the main features of behaviour being investigated. Also care should be taken in relation to correlation between two models. False correlations are not uncommon.

6.6.3.6 Results interpretation

Results should be regularly interpreted to seek to develop understanding of the behaviour of the system. This can contribute to the validation and verification processes.

6.6.3.7 Sensitivity analysis

Varying the model to assess the effect of the variations on behaviour is a very important review activity. Doing this helps to develop understanding of behaviour which can inform both the validation and verification processes.

6.6.3.8 Competence assessment

It is important to ensure that those using the software have the necessary competence.

6.6.3.9 Software assessment

Software validation: The software should be validated by asking the question: Is the software capable of implementing the model?

Software verification: The software should be verified by asking questions such as: Is there adequate evidence that the software has been checked for accuracy? Have benchmark analyses been carried out?

6.7 Modelling review process

Figure 6.7 shows a basic review process and **Figure 6.8** represents a full review process. The former would be used in a standard context that is familiar to those working on the type of model. The latter would be used in an innovative situation where there might be significant uncertainty, complexity and/or where the context is safety critical, for example, nuclear power installations. Results verification is always needed but in standard contexts validation may not need deep consideration and sensitivity analysis may not be required. In a complex innovative situation, very careful attention to all the review activities may be essential.

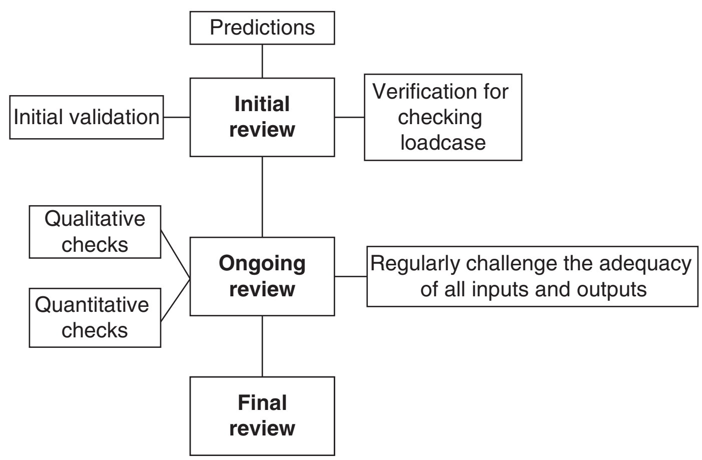

Figure 6.7 Basic modelling review process

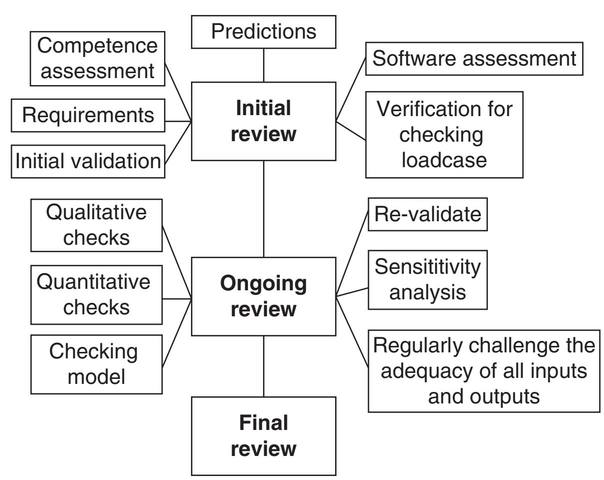

Figure 6.8 Full modelling review process

6.7.1 The initial review

Activities for an initial review may include:

- Make predictions about the expected outcomes at the outset and check these against the results as they emerge from the process.
- Define the requirements for the model.
- Assess competence and software.
- Carry out an initial validation of the model.
- Set up the data with a simple loadcase (a checking loadcase) and carry out results verification.

6.7.2 Ongoing review

- Continue with development/production runs. Do quick qualitative verification on a regular basis and further quantitative checks as appropriate. Continue to consider model validation issues if the model is altered and to interpret validation information from the results if practical.
- As appropriate, carry out sensitivity analyses to develop understanding of the behaviour of the system being modelled and to contribute to the validation analysis.
- Continually challenge the adequacy of the inputs to and the outputs from the process.

6.7.3 Final review

Carry out final versions of the model validation and results verification and record the results as appropriate to the required QA procedures. Assess the outcomes against the initial predictions and seek to identify the sources of important differences between the two.

6.7.4 Example of a modelling review for a roof truss

This example provides an example of a modelling review report.

6.7.4.1 The engineering model

Figure 6.9 gives an elevation and connection details for a roof truss. It is supported at each end on masonry walls. The trusses are at 2.5 m centres.

6.7.4.2 The requirements

The purpose of the model is to estimate the deflection and internal forces in the structure under permanent and non-permanent loading.

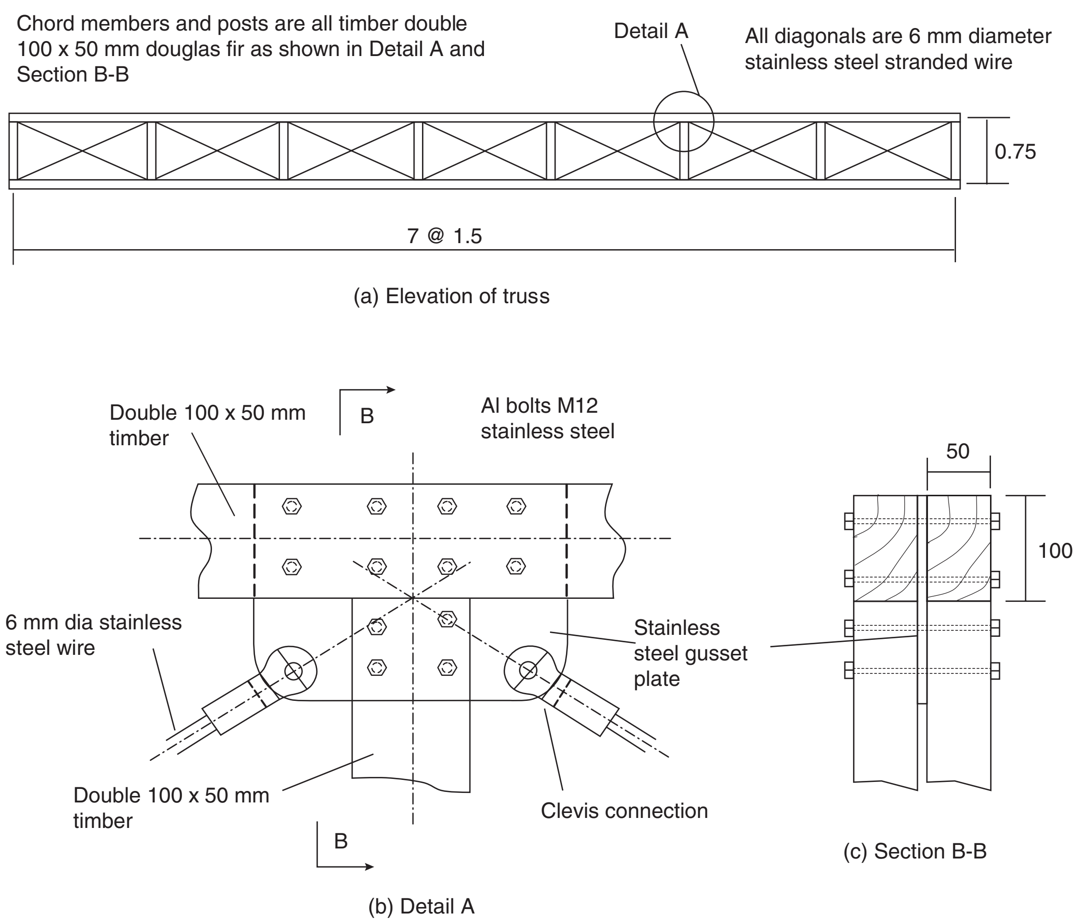

Figure 6.9 Timber truss with stainless steel bracing

Loading

Loading on roof: Permanent Load G = 1.3 kN/m²

Non-permanent load Q = 1.0 kN/m²

Design load for quoted case w = 1.35G + 1.5Q

6.7.4.3 The analysis model

Figure 6.10 shows a plane frame model of the truss.

Element types:

Chords and posts: Beam elements – bending and axial deformation – with no shear deformation

Diagonals: Truss elements – only axial deformation, no bending.

Section and material properties are given in **Table 6.3**.

6.7.4.4 Results

The following outputs are depicted: deflected shape (**Figure 6.11**); bending moments in the chord elements (**Figure 6.12**); axial forces in the chord elements (**Figure 6.13**).

6.7.4.5 Model validation

Elasticity

Timber: Acceptance criterion: design to code of practice.

Steel wire: Design to code of practice.

6.7.4.6 Element types

Beam elements for timber members: Timber members and connections to be designed to take combined axial forces and moments predicted by the model.

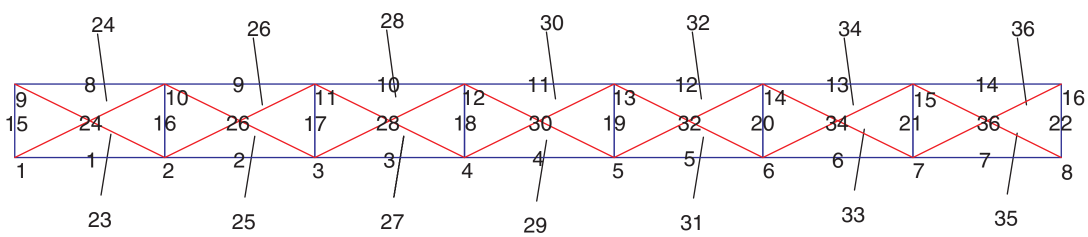

Figure 6.10 Analysis model showing node and element numbers

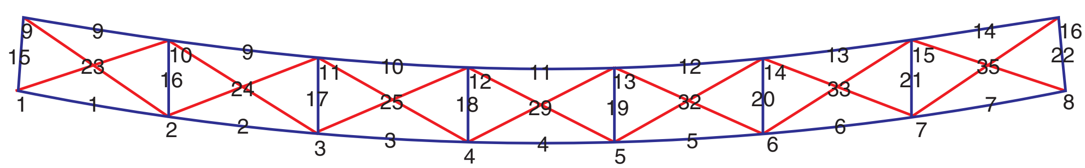

Figure 6.11 Deflected shape

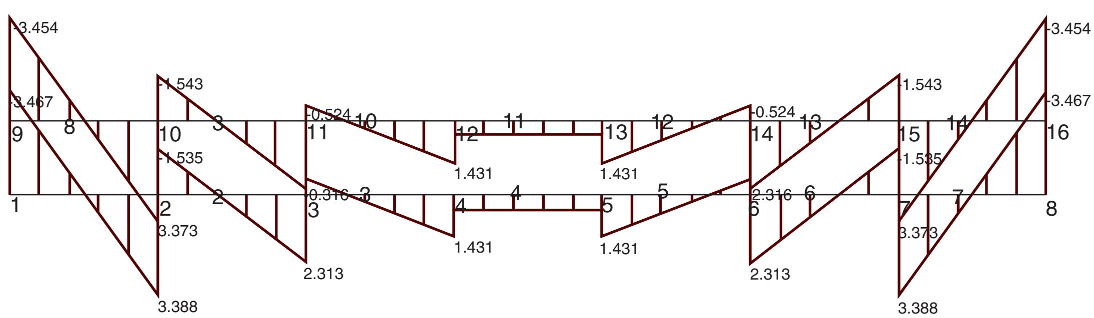

Figure 6.12 Bending moments in chord elements

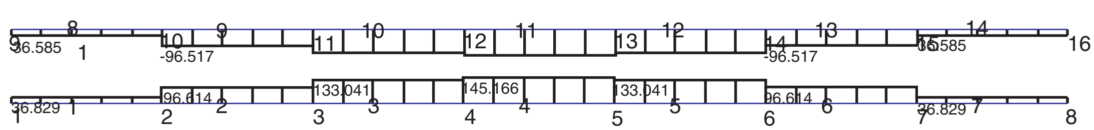

Figure 6.13 Axial forces in chord elements

Section	Area m^2	I m^4	E (kN/m²)	Elements	Element type
Chords – double 100 × 50 timber	0.01	8.33E-6	12.0E6	1 to 22	Beam
Diagonals – 6 mm diameter stainless steel cable	2.83E-5	–	197E6	23 to 36	Truss

Material: Linear elastic for all elements
Restraints:
Node 1 – pin
Node 2 – horizontal roller
Loading The loading applied at point nodes vertically downwards is given in Table 6.4.

Table 6.3 Section and material properties and element types

Nodes	Applied vertical nodal loads
10 to 15	12.2 kN
9,16	6.1 kN

Software used: Strand7

Table 6.4 Loading applied in model

Bending theory: Criterion for neglecting shear deformation: span:depth > 5 Minimum span:depth = 0.75/0.1 = 7.5. **OK**

Truss elements for the diagonals. These elements will not take compression and therefore the compression diagonals should be removed for each loadcase or use software feature to neglect the effect of compression diagonals. **ERROR**

6.7.4.7 Connection eccentricity

Member axes do not meet at a single intersection point as shown on Detail A on **Figure 6.9**. This cannot cause moments in the diagonals (they cannot take moments) and is unlikely to cause significant extra moments in the timber elements. **OK**

6.7.4.8 Restraints

The roller support at node 8 allows horizontal movement at that end. There will be some horizontal restraint at the level of the support but it is conservative to neglect it. **OK**

6.7.4.9 Euler buckling

Diagonals will not take compression (see section 6.4.7.6). Top chords and posts to be designed to code of practice.

6.7.4.10 Loading

Check values of point loads used:

Loading on roof: Permanent Load $G = 1.3$ kN/m²

Non-permanent load $Q = 1.0$ kN/m²

Design load $w = 1.35G + 1.5Q = 1.35 \times 1.3 + 1.5 \times 1.0 = 3.26$ kN/m²

Load/m on trusses = $wxS = 3.26 \times 2.5 = 8.15$ kN/m

where S is the spacing of the trusses

Load at internal panel point on truss = $8.15 \times 1.5 = 12.2$ kN

Load at external panel point on truss = $12.2/2 = 6.1$ kN

OK

6.7.5 Results verification

Data check: Nodal coordinates – checked. Element properties – checked. Loading – checked.

Equilibrium of vertical load:

Applied vertically $6 \times 12.2 + 2 \times 6.1 = 85.4$

Sum of vertical reactions at nodes 1 and 8 – $2 \times 42.6999 = 85.3999$ **OK**

(Note: value of 2.6999 from computer output. Use of a high number of significant digits for this check is recommended see MacLeod (2005), Section 3.6.2.)

Restraints:

Zero deformation in X and Y directions at node 1

Zero deformation at in Y direction at node 8 **OK**

Symmetry: Vertical nodal reactions = 42.6999 are the same at nodes 1 and 8 (Note: value is from computer output. Reason for use of high number of significant digits as for vertical equilibrium check) **OK**

6.7.5.1 Check moment equilibrium at Node 2

Sum of moments at Node 2 (clockwise + ve (see **Figure 6.14**))

$4.923 - 3.388 - 1.535 = 0.0$ **OK**

Form of results – displacements

The deflected shape (**Figure 6.11**) is curved as would be expected with UD loading. The distorted shapes of the panels show significant shear deformation of the type shown in **Figure 6.15**.

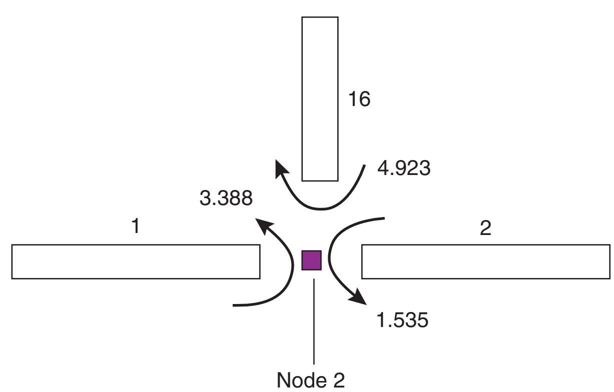

Figure 6.14 Internal moments at Node 2

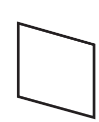

Figure 6.15 Shear deformation

Form of results – internal forces

- The chord bending moments decrease towards the centre line and there are points of contraflexure in them except for the centre element where the BM is constant. Having points of contraflexure near to the centre of the beams (as in this case) is typical of vierendeel action. A (secondary) vierendeel action component is expected to occur with this type of frame because of the moment continuity between the chords and the posts. The reason why the chord moments decrease towards the centre line is because they are mainly a function of the global shear force on the truss – a characteristic of vierendeel action.
- The axial forces in the chord members increase from the supports to the centre of the truss. This is because they act like the flanges of an I-beam: the axial loads are dependent on the bending moment in the beam (which increases towards the centre line). The equivalent bending moment in the truss is the 'global' moment – see section 6.7.5.2.
- The axial loads in the tie members are approximately equal and are opposite in each panel and decrease towards the centre line of the truss. This is consistent with them resisting the shear forces across the truss panels (the 'global' shear forces).

6.7.5.2 Checking model – internal force actions

1. Check forces in diagonals in panel next to support.

   Shear in panel = 42.7 – 6.1 = 36.6 kN

   Assume that this is taken by the diagonals equally.

   Therefore estimated force in a diagonal $N_{de} = (36.6/2)/\sin\theta$

   $\theta = \text{atan}(0.75/1.5) = 0.464$ rad $\sin\theta = \sin(0.464) = 0.448$

   $N_{de} = (36.6/2)/0.448 = 40.85$ kN

   From the results (quoted in **Figure 6.16**) the average of axial forces in diagonals

   $N_d = (30.858 + 30.585)/2 = 30.72$ kN

   % difference between estimated value and that from computer results:

   $= (N_{de} - N_d)/\ N_d*100 = (40.85–30.72)/30.72*100 = 33\%$

   The positive sign for the % difference shows that the diagonals do not take all the shear in the panel. The chord members take a significant amount of shear.

   From the results (not tabulated here) they take 4.57 + 4.55 = 9.12 kN.

   Hence the shear taken by the diagonals is 36.6 – 9.12 = 27.48 kN.

   Hence an accurate estimate of the load in the diagonals is:

   $N_{de2} = (27.48/2)/0.448 = 30.70$ kN

This gives close correlation with the computer results (30.72 kN) as it must, since it is not an approximate calculation but a full equilibrium check (to 3 significant digits) involving no assumptions except for truncation of the values.

This is an example of how doing checks can inform understanding of behaviour. In this case the chords take about one third of the shear in the panel. The proportion of the end panel shear taken by the chords is an indication of the effect of local bending in members of the truss (e.g. as characterised by M_c in **Figure 6.18**).

2. Check axial forces in chord members at the centre of the span using the equivalent beam checking model shown in **Figure 6.17**.

   The 'global' bending moment M_g (**Figure 6.18**) at the centre of the span is

   $M_g = WL/8 = 85.4*10.5/8 = 112.1$ kNm

   Taking moments about point A on **Figure 6.18** and neglecting the local bending moments in the chords (M_c – see **Figure 6.18**)

   $M_g \approx N_c b$

   Hence the estimated axial force in a chord at centre of span (b = 0.75 m) is

   $N_{ce} = M_g/b = 112.1/0.75 = 149.5$ kNm

   From the results (not tabulated here) – axial force in Element 11 – $N_{11} = 145.1$ kN

   % difference = $(N_{ce} - N_{11})/N_{11}*100 = (149.5 - 145.1)/145.1*100 = 3.0\%$

The estimated chord axial force is slightly larger than the computer model value because:

- the local moments in the chord members (indicated in **Figure 6.18**) are neglected in the calculation. As previously noted these are low near the centre of the truss and therefore do not significantly affect the axial forces in the chord in this area.
- The load in the computer model is not uniform but has been formed using discrete loads at the panel points. The global moment in the centre panel of the computer model is 109.8 kNm as compared with the 112.1 in the checking model i.e. a difference of about 2%.

Check – **OK**

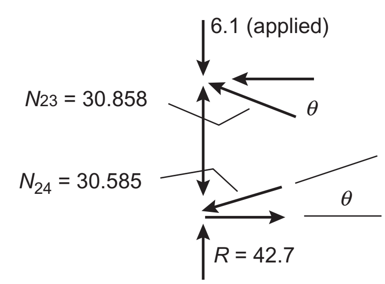

Figure 6.16 The forces in the left end panel

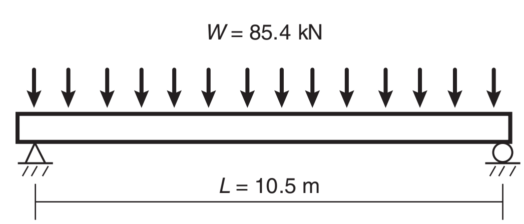

Figure 6.17 Equivalent beam model

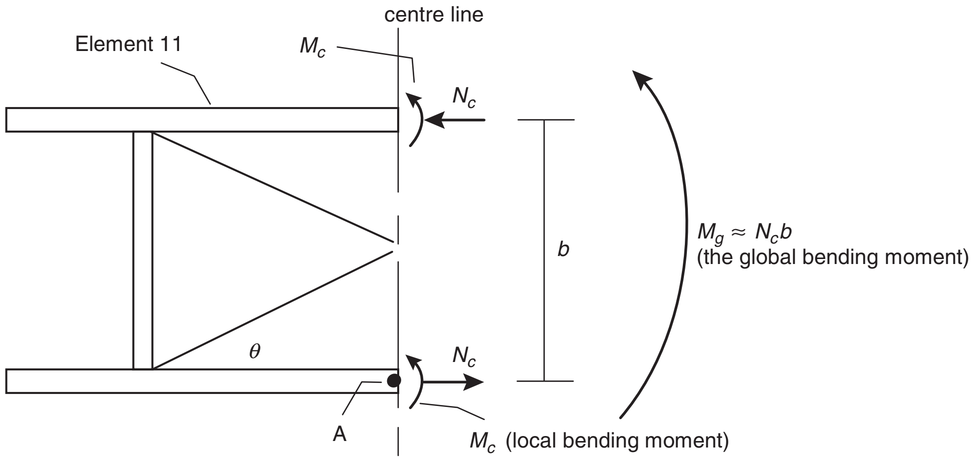

Figure 6.18 Section at centre line of truss

6.7.5.3 Checking model – displacements

A calculation using an equivalent beam which models bending deformation of the truss based on axial deformation of the chords, and shear deformation of the truss based on the axial deformation of the diagonals and the posts (see MacLeod (2005), Section 5.10.4) was carried out. This gave the following results:

Vertical deflection at the centre of the span – Δ

Checking model results:

Component of Δ due to bending mode deformation – $\Delta_b = 0.038$ m

Component of Δ due to shear mode deformation – $\Delta_b = 0.057$ m

Total = 0.038 + 0.057 = 0.095 m

Computer model result

Computer value – vertical deflection at nodes 4 and 5 = 0.0781

% difference from checking model = (0.095 − 0.0781)/0.0781*100 = 22%

Reasons for this difference include:

- Neglect of local bending action in the chords and posts in the checking model which is included in the computer model. This will overestimate the checking model result, as evidenced by the positive sign of the difference between the two models, and is likely to be the main source of the difference.
- The difference between the discrete representation between panel points in the computer model and the continuous function used for the checking model. This is likely to be small – a few per cent.
- Assumptions made to formulate the bending and shear stiffnesses of the equivalent beam – also likely to be a few per cent.

The main difference is likely to be due to the neglect of local bending. This could be estimated by including an allowance in the checking model deflection using the equivalent beam for a vierendeel frame as described in Section 5.11.3 of MacLeod (2005).

Check on displacement – **OK**.

6.7.5.4 Overall review

Model is generally satisfactory but compression diagonals must be removed.

6.8 Stability and robustness

For more detailed information see Chapter 4: *Stability* and Chapter 12: *Structural robustness*.

6.8.1 Definitions

In relation to stability and robustness the following definitions are based on ideas from Zalka and Armer (1992).

- A *stable structure* is not susceptible to small perturbations.
- A *robust structure* performs well under large perturbations.

In an unstable condition a small perturbation can cause the system to collapse. Under abnormal loading a robust structure may suffer local damage but will not suffer overall collapse.

6.8.2 Assessing stability

A good strategy is to write a stability assessment of the structure. Questions for such an analysis include:

1. Is the structure as a whole approaching its global critical load? This can be assessed from the results of an eigenvalue critical load analysis or, for buildings, using an approximate method (MacLeod and Zalka, 1996).
2. Under the design loading are any of the members of the structure approaching their critical load?
3. Is there potential for local buckling at connections/load concentrations?

6.8.3 Assessing robustness

The collapse of the Ronan Point multi-storey building in 1968 has had a fundamental effect on design for robustness (Bussell and Jones, 2010; IStructE, 2010). Assessment of the reasons for the collapse led to acceptance of the concept of 'progressive collapse' as an important feature in the design of buildings and that damage should not be disproportionate to the cause (IStructE, 2010).

Similarly to stability, a robustness analysis can be prepared. Relevant questions include:

1. Does the structure have adequate ductility?
2. Is there redundancy to allow for alternative load paths?
3. Are the components adequately tied together?
4. Are the potential consequences of any lack of robustness acceptable?

6.9 Concluding remarks

Concerns are often expressed about structural engineers doing calculations using software without adequate appreciation of what is behind the calculations or whether the calculations have been correctly implemented. Controlling the processes used in structural design outlined in this chapter should lead to a lessening of these concerns.

6.10 References

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6.10.1 Further reading

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Chapter 7

Key issues for multi-storey buildings

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CONTENTS

The success of multi-storey buildings must be judged across all design disciplines and be seen through the client's and user's eyes. The structural engineer is a central member of a multi-disciplinary team aiming to achieve the best answer to the brief. The design must develop through an ordered series of stages, with appropriate options investigated, discussed, discarded or adopted. Excellent communication is the key to shared understanding across the team.

Once the appropriate loads have been defined the building's structure should carry these to the ground in a clear and straightforward way. The appropriate materials and structural form need to be investigated and selected. On plan movement joints and stability systems are positioned, and the columns laid out. The interrelationship of the different uses within the building must be understood, the most direct vertical load paths through these being strongly preferred, with any required transfer structures identified early in the design process. Within each floor zone structure, services and architecture should be coordinated within the optimal depths. The engineer will also have input into 'non-structural' issues including partitions and cladding, fire and corrosion, plant and stairs, contributing to the success of the building as a whole.

7.1 Introduction

A multi-storey building is much more than just a multi-storey structure. It is a complex three-dimensional object with many components, all contributing to its users' perception of its success – architecture, building services, structure and fit-out. Through its lifetime, which may be longer than planned, it may evolve or even completely change in function. It requires resources to put it together and will continue to consume these throughout its life. It may be a blot on the landscape or bring joy to the world.

Within this present and future complexity the engineer needs to deliver a structural design that is efficient, safe, fulfils the structural brief and enables all the other building elements to perform as required. To achieve this wider definition of success the structural engineer must proactively work as part of a multi-disciplinary team achieving the client's needs.

This chapter starts with 'Managing the design' (Section 7.2). Achieving a successful building requires a team to go through a design life-cycle of ordered steps of analysis and decisions. Throughout this time they need to test their proposals against the client's brief, understand the needs of the other disciplines and communicate the implications of their chosen structure.

Section 7.3 discusses how the structure works within the building as a whole and the requirements the choice of structure must be tested against. The advantages and disadvantages of some common structural systems are discussed. Section 7.4 discusses key considerations when choosing the layout of columns, cores and movement joints. Section 7.5 looks at issues that drive the design through the height of the building, both overall and floor by floor. Section 7.6 discusses the way the structure must work with 'non-structural' requirements and Section 7.7 looks at how all these issues develop and drive our design as we strive to build higher.

So when is a building 'multi-storey'?

Accommodation on just the ground floor with just a roof and walls evidently isn't and anything more than that probably is. However, as more floors are added the vertical circulation (stairs, lifts, escalators), service risers and horizontal stability structures begin to increase in importance for the design, and their impact on the relationship of the various building functions increasingly needs to be coordinated in three dimensions.

The building will be supported by a foundation and may sit over a basement structure. The configuration and movement of these may well influence the superstructure above. However, discussion of these is beyond the limits of this chapter.

7.2 Managing the design

7.2.1 The design life-cycle

In order to deliver the optimal design of building when judged against cost, quality, time or environmental impact, it is important that the design team proceeds in an orderly series of steps: the design life-cycle. First they need to understand the brief, investigate and evaluate the available options and then make the best choices. After that the information for costing, tender and construction needs to be developed.

Terminology around the world varies but the work of all building design teams should go through a series of generic stages. The exact balance of work carried out and the split of deliverables between the stages varies slightly across projects, contracts and regions. However, this chapter follows common UK building terminology, calling the stages in order:

- Brief definition – understanding the problem.
- Conceptual design – investigating the options and their implications.
- Scheme design – choosing and laying out the best option.
- Detailed design – producing all the information needed for construction.
- And finally, support for the construction process – responding to problems experienced on site.

Common design life-cycles in the UK include those defined by the RIBA (Royal Institute of British Architects), the CIC (Construction Industry Council) and, for rail-related projects, GRIP (Guide to Railway Investment Projects). The structural team needs to understand the requirements of the system being used by the project, the level of decision-making defined at each stage both for structure and the other disciplines and the requirements for their deliverables at these phases.

Design teams are sometimes tempted to try and jump straight to a final form without a process of investigation, when they narrow the options over time by increasing detail and certainty. By doing so they are likely not to have understood the key structural and multi-disciplinary design drivers and will deliver a non-optimal solution. Often an over-hurried choice of solution will require the design to be changed later to accommodate issues missed due to the lack of investigation during concept or scheme design.

Figure 7.1[1] shows the cumulative number of hours that the structural design team might input into the design and delivery of a building over a two-year process. It also shows how the cost of any change to the building, be it design or construction cost, increases during this period. Properly executed concept and scheme stages minimise the potential for later unnecessary and disruptive costs as well as delivering a better building.

Whilst clients often recognise the need for an architect to commit resources to the brief definition and conceptual stages they are sometimes tempted to reduce the time and fees available for the engineers during this period to almost zero in the belief that they will add little value at this early stage. There is no doubt that an experienced architectural team can anticipate many of the structural issues, layouts and zones for a 'conventional' building but it is important that the future structural team has 'buy-in' to these decisions, especially if there are any unusual aspects to the project – and there often are.

The key issues that the structural engineer needs to investigate, agree and define early in the design process are the ones that affect cost, coordination with the other disciplines, the programme and the procurement route. In particular, during concept design these key issues might include:

- The loads to be carried.
- The basic layout of the columns, defining the typical bay size.
- The need for any transfer beams beneath columns.
- The size and support of any cantilevers.
- The depth of the zones available for the structure at each floor level.
- The arrangement on plan of the stability systems and whether these can be moment frames cores with bracing or stability walls.

It can be seen that at concept stage these decisions do not necessarily prescribe a specific solution type. Instead, they allow the space and cost that will allow a reasonable, optimal solution to later be developed at scheme design which can then go on to be successfully detailed and then constructed.

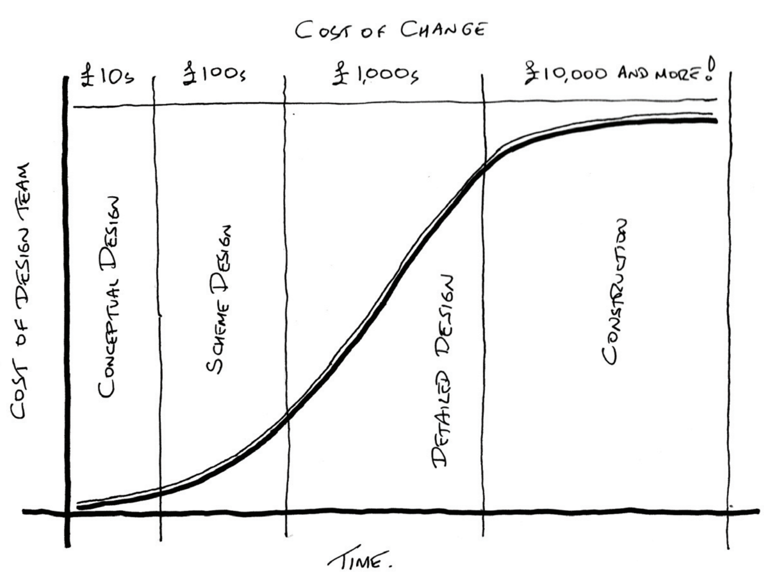

Figure 7.1 Structural design time and the cost of change

Whilst the team may have developed some preconceptions and preferences for the structural type during concept design, it is at scheme design when different structural options will be looked at and evaluated against one another. Concept design should have defined the structural issues that need solving. Scheme design will choose the systems that will be needed. Detailed design will finalise the configuration of these systems and deliver all the drawings, specifications and quantities that will be needed by the contractor to construct the building.

Often events such as delays to receipt of information or decisions or changes to previous assumptions will mean that some aspects of these activities spill over into the next phase, but it needs to be recognised by the team as a whole that this has the potential for disruption, delay and cost each time it occurs.

If a building design team understands the client's brief, studies the right options at concept design, agrees and records decisions with the client and then develops, refines and confirms these at scheme design, it will be in a position to use the detailed design stage to produce the final drawings and specification almost without further input from the client. Early value engineering can be used as part of this process to refine details and optimise specifications, rather than lift the lid, late in the day, on options that should have been properly considered in the concept and scheme design stages, with the resultant disruption that flows from late change.

Pressures from the wider project will complicate this design life-cycle, but by focusing on progress through these decisions, with some appropriate flexibility, the structural team will achieve a better result for all team members, and deliver a better structure and building to the client.

7.2.2 Understanding and anticipating the brief

Clients wanting a multi-storey building do not actually want a structure. Their priority is the functions they want to carry out inside the building. If someone invented an anti-gravity machine that could effectively and cheaply keep carpets apart the structural engineer would be out of a job! Given that the structural engineer's role is to provide a currently necessary evil it is therefore important that they properly understand what the client actually wants so that the structure does not compromise these aims and literally and metaphorically supports them.

Often the engineer will focus on the structural implications of the architect's design. However, it should not be forgotten that this is already one step removed from what the client actually asked for – it is the brief viewed through the architect's eyes.

The structural team needs to have its own clear understanding of:

- What does the client say they need?
- Do they have any particular priorities?
- Do they have needs not stated?

The list of implications that jump out of the brief can be different when read by the different members of the multi-disciplinary team, so it is important that the structural engineer understands and voices their opinion as opposed to focusing on the implications of the architect's view.

Sometimes there are direct client instructions and a clear brief, but often the client's needs and aspirations for the project emerge during meetings and correspondence during the earliest project stages – the client's own view of the project will often be evolving at the same time as that of the team. This is one of the reasons why the structural engineer should be proactive around the project table as early in the design process as possible, allowing a first-hand understanding of developing client thoughts. As the brief develops, the engineer should 'reflect it back' in writing to the wider project team to make sure there is a shared direction.

During these early project stages and throughout the project it is worthwhile for the structural engineer to remind themselves that they are the structural expert and therefore should lead the structure's development. Often the client and other team members will believe that they fully understand the subject – all humans deal with gravity on a daily basis and believe they have an appreciation of what steel and concrete are. Many project managers come from a structural background and architects love to play with structural forms. They all probably think they know the answers.

They may know some of them, but most likely not all. A proactive engineer will build on this basic understanding with enthusiasm as we all like subjects we think we understand, and will engage with the team regarding the structural issues faced. A structural engineer as team teacher can promote a fully integrated approach across the wider project and this will lead to a better final building.

A subject that is likely not to be explicit in the client's brief is the external constraints posed by the site. The strength and stiffness of foundations, external sources of vibration, buried services or archaeology, road and rail limitations, adjacent slopes and construction access are among the constraints that need to be understood by the whole design team as solutions are investigated. The structural engineer is the natural leader in these subject areas and may need to define the investigations and decision processes required.

Where multi-disciplinary design teams are working beyond their previous experience, in a new geography or on a building type they have not previously constructed, the engineer should also play a role as the brief's structural interpreter to the team. In areas with high winds, severe corrosion and, in particular, earthquakes, the final structure will impose greater constraints on the multi-disciplinary building form and an understanding of the structural needs across the team will greatly reduce frustrations, problems and rework later in the design process.

7.2.3 The brief's impact on the structural form

Several aspects of the brief and the concepts developed by the team in response, either explicitly or implicitly expressed, will have particular impact on the final structural form.

'Flexibility' is much desired by many clients and often sold to them by design teams. However, flexibility comes at a price. It means that spans between columns become greater, increasing the weight of the beams. It means that structural walls are eliminated, compromising efficient stability strategies. It means that beams are forced to be shallower, further increasing their cost.

Some flexibility is important – things do change in the future! However, it is important that the team understands the cost of the flexibility it is including and justifies this as the right choice for the client. Was it right that I designed a school in the mid-1990s with heavy concrete portal frames for every span that gave the future ability to move the partition walls between classrooms? I doubt any of the subsequent headteachers have chosen to move the blockwork partitions with the summer chaos that would cause. Since the classes all contained around 30 children, why would the teacher want one for 60 or a 40/20 split in the future? The architect was convinced this was needed and the client (future teachers – not their own money being spent by the way!) loved the idea. I lost the argument and the structure became much more expensive as a direct result of future flexibility that will probably never be used.

Defining the loads to be supported by the structure is a key activity for the engineer and is surprisingly often not given the attention it deserves early in the design process. The structure is there to support loads, so literally everything flows from how big these loads are.

Whilst it is important that the loads chosen in response to the brief are safe, the temptation to be over-conservative should be resisted as it will implicitly lead to a less efficient structure and building. High loadings are sometimes chosen to give the 'flexibility' discussed above but often without teams understanding the multi-disciplinary and cost implications. Sometimes they are chosen by the engineers to reduce 'design risk' from later changes – the structural team should ask themselves if that is the right approach, or whether that has been done for their own convenience.

In particular, early decisions need to be fixed in the brief and agreed across the teams about screeds, finishes and partitions. These superimposed 'dead' loads are often primarily controlled by the architect and can add up to be greater than the imposed loads from the building's functions. Replacing screeds and hard finishes with lightweight partitions of blockwork can result in big structural savings. However, these need to be agreed and fully 'bought-into' by the client and architect as later changes will cause major structural rework and knock-on effects across the disciplines.

Finally any assumptions about the structural form by the client or architect need to be understood as early in the design's development as possible and investigated to establish feasibility and reasonableness. An architect's love of structure can sometimes lead to 'expressed' solutions that are inefficient compared to the engineer's optimal response, and this inefficiency should be explored, established and recognised early in the design process. A desire for exposed structure within the building can lead to savings in finishes and improvements in thermal control, but will drive the structure towards certain solutions. The other disciplines need to be committed to the overall 'exposed' concept as they also need to work towards achieving efficiency from the adopted structural strategy.

The discussion above re-emphasises the need for the structural engineer to communicate, explain and teach the team the brief's structural implications for the early design decisions. These issues have to achieve their proper weighting within the project's priorities. Although they are subjects of key importance to the structural engineer they are not necessarily the top project issues. However, the multi-disciplinary concept developed by the team must properly respect and accommodate the structural requirements.

The building concept developed in response to the brief needs to deliver a realistic, agreed and recorded amount of future flexibility that allows future refinement of the design and appropriate future client change. This planned response will be much more efficient than resorting to over-specifying loads, zones and sizes in the undefined hope it all doesn't need to change later.

A clear, simple, understood and agreed solution will be best. Things will get more complicated during later project stages, so beginning with a clear conceptual strategy is vital.

7.2.4 Communicating the structural concept and scheme

A building needs to be considered in three dimensions as a series of interrelated multi-disciplinary systems sharing the same space. Each system contributes to the success of the building as a whole and is designed by a different discipline, but they cannot work blindly ignoring the presence of the other adjacent systems. The primary role of the architect is to coordinate these into a completed and successful whole, but all team members play their part in achieving success not only in their own terms, but for the team as a whole.

Each discipline looks at the building through different eyes, reflecting their training, preoccupations and responsibilities:

- The architects see a series of surfaces defining spaces, but beyond that they see the way that people will flow through the building.
- The building service engineers see behind the surfaces to where parallel systems of wires, air ducts, pipes in and pipes out radiate from key plant spaces. However, beyond that they see the flow of water, electricity, heat and cooling and increasingly the natural airflow passing through the building spaces.
- The structural engineers see grids of beams made rigid by slab diaphragms. These are spaced apart by columns and stabilised by cores or bracing through the building height. But beyond these physical elements they see gravity, wind and seismic forces flowing from one element to another and eventually to resolution in the ground.

Because they see through different eyes, and hence see these different flows, the diagrams they each draw to express the way the building works all differ. Architects are taught how to communicate visually and their sketches are often seen during the project. However, the engineers should also have the confidence to produce and share diagrams of the way they see the structure working, which is very different from what the structure is. These sketches are an invaluable way to promote understanding across the team and steer the building towards becoming efficient multi-disciplinary whole.

Many engineers avoid tabling their diagrams as they feel they are not as polished and sophisticated as the architect's. However, architects will invariably love working with an engineer who doodles sketches of various ideas and worries. They allow the team to see the building in a new light, promote understanding and hence ease their job of coordinating the multi-disciplinary whole.

On several occasions I have made physical models to explain a structural issue and taken these to meetings. To me, they look rough and unsophisticated compared to the architect's 'professional' models. However, a structural engineer builds a model of how the structure works, not how it looks. Architects love to get their hands on this kind of model. You hardly ever get them back and on one occasion the model ended up on display in the architect's reception. The structural concepts are at the heart of the project when you have achieved that!

As discussed earlier, all design processes, not just structures or buildings, must go through the following simple generic steps once the brief is defined and understood:

- Review the options available – concept design.
- Select a solution – scheme design.
- Produce information required for construction – detailed design.

The size of team and the resulting number of design hours spent on the project ramps up through each of these stages, perhaps being 5%, 20% and 75% in turn. This split will vary from project to project. Concept and scheme design should involve a larger percentage of experienced, and thus more expensive, engineers than later stages so that the implications of key decisions are fully understood, and their availability for input and review is key to the success of these phases and the project overall.

Despite being a relatively small proportion of the project costs, the importance of concept and scheme design to the design as a whole cannot be over-emphasised. The primary cause of inefficient design processes and inferior final deliverables is the failure of these first steps to deliver a clear, robust strategy that allows the detailed design to proceed smoothly and with confidence. As the definition of the design increases through the project the implications of any change become greater, as discussed earlier.

Concept design is the moment to get as many ideas as possible out of people's heads that can be evaluated by the team. I love that moment when the paper is blank and the project's design could turn in any direction. Other people hate it and need more decisions to be made to constrain the solutions and channel their thoughts – but their moment will come later in the project when it needs to be delivered!

Make sure that a wide variety of structural solutions covering all key strategies are generated during concept design and put down on paper:

- Concrete, steel, timber, composite?
- Long span, short span, arched, suspended?
- Spanning north/south or east/west, waffled or diagrid?
- Integrated with or separate from services?
- Architecturally expressed or hidden?

Solutions do not need to be fully developed into a complete building. Often, a typical bay or simplistic stability diagram is sufficient. It does not matter at this stage that an idea has obvious shortcomings that have not yet been answered. Often, the generation of so many parallel ideas will begin to suggest cross-over solutions and it is quite likely that the favourites that emerge will be hybrids of several initial ideas.

However, it is important that this process of investigation is documented, perhaps with every idea being given a 'concept number'. These concepts can then be explored with simple hand calculations, reviewed and evaluated.

This documentation becomes an important point of reference later in the design process when 'new' ideas appear as a magic answer to the developing complexities of the selected solution. Evidence that a similar idea had been previously looked at and rejected is important to keep detailed design efficiently on track. A new idea at a later stage will often look more appealing than the developing design as its nasty complexities have not yet had a chance to appear. Late switches to new solution strategies are often regretted as they are often not thought through and later generate a new range of problems.

Documentation of the conceptual thought process also demonstrates the value added by the structural team at this early stage. At concept design, the structural team often has to deliver relatively few sketches, which in themselves do not suggest much work has been done. By backing these up with documentation of all the ideas looked into and rejected a client and team will better appreciate the value the structural engineer has brought to the concept and the amount of thinking behind these early decisions.

The structural team's deliverables at concept design will vary according to project requirements and local practice. However, there is likely to be a concept design report which will:

- Record the key aspects of the brief.
- Discuss and define the key structural design drivers such as loads, available zones and market conditions.
- Define and compare the pros and cons of a number of structural solution types.

It may be that a preferred scheme is identified, but this is not a requirement until scheme design.

Before the advent of today's analysis and computer-aided design (CAD) tools, concept design was often presented with hand sketches of typical structural bays and stability diagrams, supported by simple hand calculations of typical members and overall building behaviour. These new technologies are sometimes to the detriment of the concept design process as they are tools more suited to the detailed design phase and suck the engineer's thoughts into too much detail too early. There is little value spending time constructing full 3D analysis models at this early stage and the team are better served by a mixture of hand calculations, 2D analysis and perhaps 'stick models' for the tallest buildings.

Modern visualisation tools have radically enhanced the ability of architects to present their ideas to the client at early project stages but it is to be noted that they often support them with simple hand diagrams. We are all sophisticated consumers of multi-media and the structural team must present their ideas in a similar mix of formats if they want their input to have its proper weight in the decision-making process. Presenting some 2D CAD drawings, not yet worked up with the detail they will have after detailed design, is unlikely to convince the team of the sophistication of your concept design. Often the way you communicate is as important as the messages you are giving.

Conversely, do not get seduced by the 'finished' appearance of modern visualisation techniques. Clients and other team members will often be drawn towards solutions that are beautifully rendered, looking as though they are complete. The fact that it is an attractive picture does not mean it can stand up. It is important that the structural engineer provides the team with the occasional much-needed dose of reality so that the team selects the optimal solution, not the best image!

At the scheme design phase there will normally be another, and more detailed, report. This will describe the final selection of the chosen scheme, but will then focus on a description of that scheme and its structural and multi-disciplinary implications. In parallel with the report there will be a set of CAD drawings defining the position and anticipated type and size of the majority of members. These drawings may well be at a smaller scale than will be delivered at detailed design and will not be as fully annotated.

7.3 The building structure as a system

7.3.1 Loads

The function of a structure is to transfer loads. Multi-storey buildings normally comprise a series of horizontal planes, the floors, occasionally with inclined ramps or roofs. These are where the majority of vertical loads occur.

Normally the building is wrapped by an external enclosure which is where the horizontal wind loads act. The mass of all the building elements can have additional seismic horizontal loads acting on it.

The floor systems span between walls and columns which, in addition to holding the floors apart, provide the horizontal stability needed by the structure and sometimes support service cores and stairs.

Whilst the shape of a building can in theory be any regular or freeform shape, the majority of new buildings are rectilinear for the practical reasons of cost, ease of construction and usability, and reusability, of internal spaces.

There are three main types of loads to be transferred:

- Vertically acting or gravity loads, including:
  - live loads imposed by the building's functions,
  - superimposed dead loads from non-structural building components such as floor and ceiling finishes, cladding and internal partitions, and finally,
  - self-weight of the structure itself, which in some long-span and concrete options will be the largest load to be carried.
- Lateral or horizontally acting loads, including:
  - wind loads,
  - seismic loading, in areas where this is a significant risk or for particularly sensitive building uses, and
  - notional horizontal loading, which is sometimes mistakenly seen as a robustness requirement but actually represents the potentially real horizontal forces that are required to stabilise the columns and walls when they are not perfectly vertical.
- Soil and water loads, which can have vertical and horizontal components.

Since the function of a structure is to transfer loads it is therefore essential that the structural engineer confidently establishes, records and agrees the size of these forces as early in the design life-cycle as possible. The combinations of these that need consideration must be established with regards to both the code requirements and reality, and sometimes reality is not fully covered by a code. Later changes to these forces or their combinations will invalidate much work that has gone before, resulting in delay and frustration. Early time spent accurately fixing and recording the forces is very important and will pay dividends later.

The live, wind and seismic loads all have some degree of dynamic component, but in the majority of cases codes increase the static forces to represent this, and the building structure normally only requires a static analysis. The dynamic behaviour of the structure only needs to be considered separately when there is a particularly flexible structure, sensitive function, loads from heavy vehicles such as trains or in extreme conditions such as the world's most earthquake prone areas.

Soil and water loads can be of very high magnitude compared to others, especially as the depth retained or supported increases. As mentioned earlier, this chapter focuses on superstructure issues and not substructures. Most underground structures have a balance of forces on either side and thus the requirement is the transfer of these forces through the structure from one side to the other rather than their support. Conversely when a superstructure has soil or water loads acting on it often this is from one side only, alongside a retained hillside, for example. This unbalanced force is likely to be much larger than all other loads on the structure and its magnitude needs to be established early on as it will drive many aspects of the design.

7.3.2 Load paths

All loads must have a load path. Before decisions are made on the form of the structural elements the designer must be clear what role these elements play in the building's load paths.

A structural designer is someone who chooses the load path along which loads flow from their point of application to their point of support. They investigate options for this path and choose an optimal configuration. They must know the load path of every load applied and load path diagrams demonstrate this understanding.

A structure transfers loads through load paths that span to points of support. Buildings are three-dimensional objects, and the load paths can be so as well, but often load paths through parts of a structure can be considered as two-dimensional plans or sections. Simpler 'one-way spanning' solutions often give easier design and straightforward construction but the efficiencies of 'two-way spanning' should also be understood.

It is important that clear diagrams are produced early in the design life-cycle that show the way that loads flow through the load paths to support at the ground. These diagrams are an important way for the engineer to think through all the elements and connections along the path that will need detailed design later. They also importantly convey to other team members the way the structure works, letting them appreciate the impact on the design if things later need to change and move. Anything that interrupts, cuts or deviates this flow of force will need to be thought through and justified or modified.

Although seemingly a statement of the obvious it is important to emphasise that all load paths must be complete. Simple examples of incomplete paths are shown in **Figure 7.2**. There must be a capable member or connection at every stage of the load's journey and this must be a key focus of early technical reviews as this is often where problems occur. Load path diagrams are a key tool in such reviews.

Computer tools now allow an engineer to build a three-dimensional structural model at an early stage, apply loads and then let the computer establish how the forces flow through the structure. However, this is an extremely dangerous practice as the engineer has no idea of whether the results they are given are either correct or appropriate. The computer is fully in charge and the operator has lost control of the design. 'Designer' would be the wrong word for their role here.

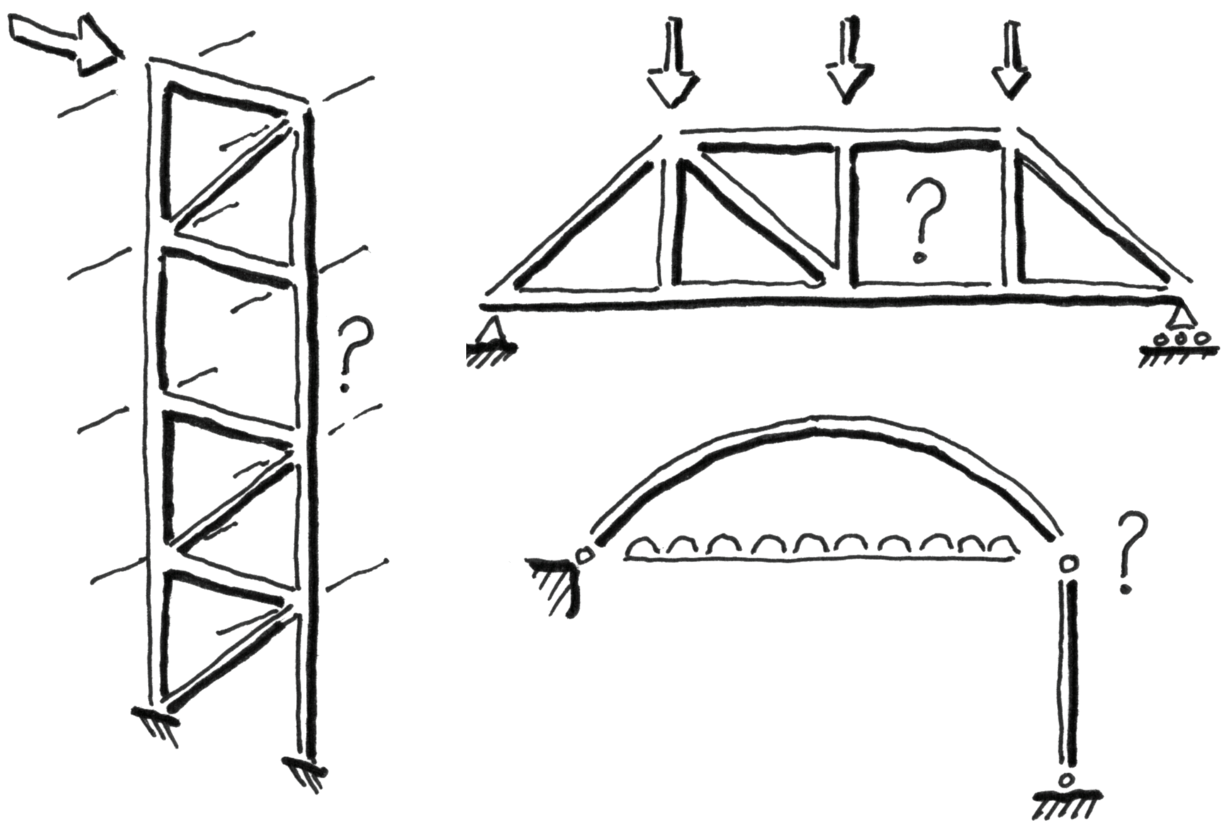

Figure 7.2 Incomplete load path diagrams

In general the most appropriate structure is likely to be the one that carries its loads most directly to its supports. This naturally reduces the amount of structural material needed to carry the load and will reduce the number of connections it passes through, reducing complexity. However, a variety of non-structural requirements (for instance, circulation, servicing and aesthetics) will mean any optimal load path diagram is likely to require local or global modifications, as shown in **Figure 7.3**. An optimal structure occasionally needs to be sacrificed to achieve an optimal building.

It is often worth any reviewer paying extra attention to the horizontal load paths as these are often more complex and less obvious than the vertical ones, with more three-dimensional behaviour and a variety of structural elements and actions. Loads are often first applied horizontally to slabs that must span between supporting elements using diaphragm action. Particular attention must be paid to the position of holes which can prevent this action taking place and to the transfer of forces from the horizontal to the vertical members, as seen in **Figure 7.4**.

7.3.3 Testing the structure's requirements

Whilst defining the load paths the engineer needs to be constantly assessing the influence the design choices being made

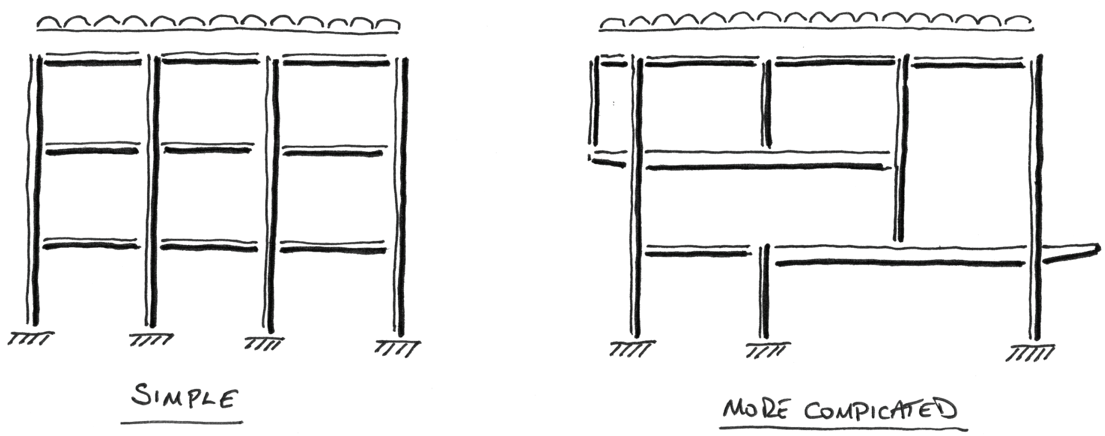

Figure 7.3 Load paths for vertical loads

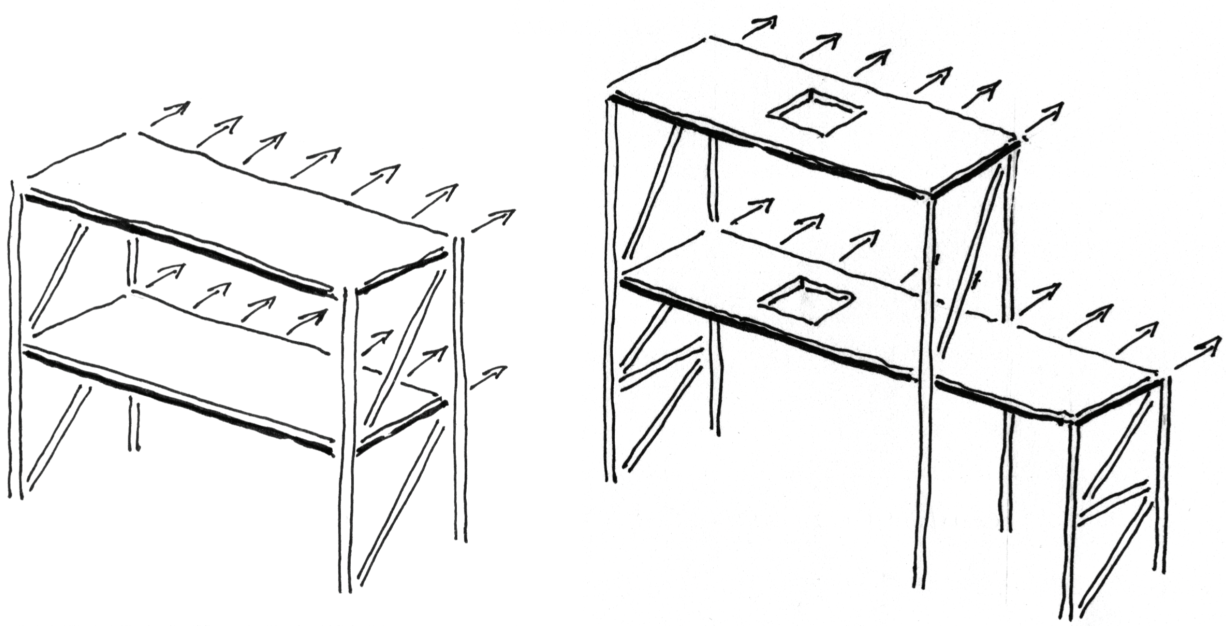

Figure 7.4 Load paths for horizontal loads

are having on the building's many constructional, functional and non-structural requirements. Different choices, configurations and combinations of vertical and horizontal structural components and materials should be considered. Different solutions have different pros and cons – and no one solution will be best for all issues. The engineer must assess these and explain them to the team so they are understood in the context of the whole building.

Most of the issues listed below relate to the horizontal structure – slabs and beams. Normally there is less material and cost in the vertical structure (walls and columns) and, other than the spans between vertical elements, this has less influence on the overall solution than the choice of floor systems.

When developing an overall solution the structural engineer must consider the following issues amongst others. There is no particular order of importance:

- The ability of the functional framing to provide the load transfer and stability needed by client's functional requirements, whilst achieving the spans and spaces required by the building's use.
- The structure's integration with the building's services distribution, both now and in the future, and in particular the ability for services to pass under and through beams, slabs and walls.
- The structural cost, both initial and over the whole life of the building, and the cost impact on other building elements.
- The control of deflections, their effect on the building's occupants and their impact on other building elements including cladding, lifts, escalators and partitions.
- The structure's response to footfall, impacts and other sources of vibration and the acoustic performance of the building.
- The way the structure and supported elements interface with the building's boundary elements at internal joints and where they come into contact with the soil and meet neighbouring buildings.
- The durability of the structure against corrosion and fire and its robustness against accidental damage.
- How safe is the structure to construct, maintain and demolish, taking into account aspects such as the need for work at height and the health issues raised by the materials and methods used?
- Ease of construction taking into account site constraints, the programme, off-site fabrication, the number of contractors required and their interfaces. All this is sometimes called 'buildability'.
- The environmental impact of the building including its embodied energy, recycled content and requirements for future maintenance. The structure's ability to replace 'non-structural' elements should be considered as well as the contribution of its thermal mass to the energy performance of the building.
- The aesthetics of the structure as expressed in the details of the finished building and in particular the surface quality of exposed structural elements and the structure's integration with other finishes.

7.3.4 Structural materials

The key step change in the history of structural engineering must be the move from 'natural' materials (earth, stone and wood) to 'modern' man-made materials, in particular, steel and concrete. Wood has recently undergone a modern reinvention with new forms allowing greater spans and reliability, with minimal carbon footprint, but it is unlikely to ever achieve the spans, load capacity and wide adoption of steel and concrete. New materials continue to emerge, for instance carbon fibre sheets, but to date they have not yet found application in other than niche markets. For these reasons in this chapter I have chosen to limit the discussion to steel and concrete solutions although much that is said could have wider application.

When deciding which structural material to use it is important to understand the local building culture, codes and skill sets. In many countries it is not feasible or indeed possible to build, for instance, a composite steel framed building of the type that has dominated the London office market since the 1980s. The UK has benefited from a history of innovation and strong competition from steel and concrete designers and contractors which means it is important to carefully study the pros and cons of the options available. Sometimes it is tough to decide which scheme is 'optimal' for the criteria chosen for comparison by the design team.

The choice of structural form needs to be carefully studied, with particular regard to what the local market can supply. Only prestige projects, with budgets to match, can source structural materials from wherever they want, and the impact of transport on costs and the environment encourages most projects to shop locally.

Key considerations for steel frames include the availability to fabricators of welders certified for more complex configurations or whether the fabricator has strong preferences for simpler 'cut, drill and bolt' connections. Is on-site welding feasible? It is strongly disliked by contractors in the UK but is the norm in California for instance.

Concrete, being a heavier material, will tend to be sourced from a much closer radius than steel. What is the ability of the local market to deliver precast, pre-stressed or post-tensioned solutions? Do not choose an option if the likely contractors cannot deliver it!

Once the possible structural forms (and mixes of forms) have been established the engineer should choose some representative bays and sections and compare the solutions, sizes, costs and implications of each, taking into account their wider multi-disciplinary impact. Scoring systems of their various merits can be used to guide debate, but ultimately the team must use their judgement to decide which structural scheme should be taken forward. It is important to know how 'representative' the typical bays studied are and whether there will be 'non-typical' conditions that will prove problematical for some solutions.

Some of the key issues that should be considered as schemes develop are outlined in the next sections.

7.4 Achieving the right structure on plan

7.4.1 Positioning movement joints

During the conceptual design of a building it is unlikely the structural engineer will have much influence over the shape of initial floor plates generated by the architectural team. Required set-backs, site boundaries, lines of sight, rights of light and other functional needs will sculpt the volume to be enclosed and the first estimates of floor to floor height will define their level and hence their possible shapes. However, once these 'first stabs' are available the structural engineer can swing into action with ideas, comments, tweaks and refinements.

Consideration of all the issues discussed below needs to happen concurrently as they are all interrelated – the position of cores influences the position of movement joints for instance. Conceptual design is an iterative process when the relationship of the variables is explored to find an 'optimal' balance – structurally and non-structurally.

A key first issue to address is whether any movement joints are needed through the structure to control horizontal movement, differential settlement or cracking and deformation. Whilst several structural sources define maximum lengths between joints, it is my experience that stretching these distances a bit further provides great advantages for the building as a whole at the cost of a little extra structural work. Structural joints have a significant impact on the performance of other elements of the building. They increase the chance of leaks, require joints in pipes, ducts and wires, and pass through horizontal and vertical finishes. If you really need a movement joint to pass across the front facade or through the entrance foyer it is best to give the architect the bad news as early as possible, especially if you want to work for them again.

The structural engineer should make a realistic estimate of how wide any movement joints need to be, taking into account any filler boards, fire seals and drainage features that may need to be inside the joint, limiting its ability to move. Joint widths are often underestimated by the project team at concept and scheme and come as a bad surprise later when properly detailed. In seismic areas, it is not only movement from wind, temperature and shrinkage that needs to be taken into account but the joint must allow for the nonlinear deformation of the building portions on either side of the joint to avoid pounding occurring. Through careful, reasonable choices of temperature and shrinkage values and assumptions, and sometimes the specification of pour sequences, it is possible to eliminate or minimise the number of joints and their widths. Careful consideration of the position of holes and acceptance that some areas of structure may need extra reinforcement or careful detailing to control cracking will provide great value for the building as a whole and its long-term performance.

I have seen the positions of joints not being fixed at concept design by a surprising number of design teams, but for me it is a key early issue to define. Quite often the other team members will not want to look at the implication of joints in detail at this early stage, but will rapidly assume there aren't any if they are not told! Making sure the team is aware of them will make for better relationships and smoother progress later in the design process.

The type and distribution of the horizontal stability systems has a significant influence on the ability to position movement joints in desirable places. The two extremes of systems to choose between, or hybridise, are firstly using concentrated structural cores with concrete or masonry walls or steel bracing, or alternatively using more distributed systems of columns and beams acting as moment frames. Stability must of course be provided in both the orthogonal directions and one common hybrid is for walls to be used in one direction and moment frames in the other.

If you are thinking of a single main stability element it is generally best to position this towards the centre of the floor plate as this minimises the twist caused when resisting uniform lateral loads. A key judgement is whether a single structural core is of sufficient width to resist the twisting forces caused by non-symmetrical loads.

With two cores within one floor plate it is highly likely there is sufficient torsional stiffness to resist the application of non-symmetrical loads. However, the structural designer then needs to consider the trapped forces that expansion, contraction or lean inducing settlement can cause between the cores – the further apart these stiff cores are the more the floor plate is prone to fight these movements.

If using moment frames distributed throughout the floor plate a building often behaves more straightforwardly when resisting expansion, contraction or twist on plan. However, as floor plates become longer in any or both directions an eye must be kept on the additional forces the extreme frames may pick up. Non-symmetrical horizontal forces can be the critical load cases for the end moment frames for long floor plates, and the bending and shear forces induced by expansion and contraction may need to be carefully looked at, especially for the columns between ground and first floor near the furthest facades.

If relying on cores to supply lateral stability, you are again somewhat dependent on the architect's choice of their location. Walls or bracing are normally located around stairs, risers and bathrooms – all areas that, like stability systems, are best run consistently from top to bottom of the building. The structural engineer needs to pay careful attention to the way the configuration of spaces inside the service cores develops and in particular the straight lengths of wall available for the stability systems. The position of doors can have a significant effect on the configuration of diagonal steel bracing or forces in the concrete wall panels around them.

The lateral stability for each section of the floor plate showing the load paths and resisting systems is a key plan diagram to draw early in the concept design stage.

7.4.2 Column layout

So, you know the size of the floor plate, how it is divided up structurally between movement joints and the position of the structures providing the building's horizontal stability. The next key issue is the position of the columns, the size of the bays they define, and the layout of the beams within them.

Columns remain a necessary evil. It is unlikely that any member of the team other than the structural engineer actually wants them – they are a useful place to put signs, but that is the only thing the architect might miss. Simplistically, the architect, and the interior designer in particular, would love you to deliver a 'column-free' space, meaning they can do whatever they want and you have delivered infinite flexibility.

However, 'column-free' space comes at a cost. As the distance between columns increases, the depth and cost of the floor systems needed to span between them also increases, complicating service layouts, increasing the depth of ceiling zones, increasing the floor to floor heights and pumping up the overall height and cost of the building. During concept and scheme design the structural engineer must explore issues with the rest of the design team so that the pleasure of column-free space is properly balanced with the pain of the increases in floor structures.

Usually the building is best served by achieving a regular grid of columns with repetition of bay sizes and dimensions. This allows easier internal planning for all disciplines now and in the future and gives the internal spaces and exterior appearance a sense of order. However, site constraints and brief requirements, particularly in congested urban sites, mean that a completely regular grid is hard to achieve, but it is worth aiming for as much order as possible.

When setting a grid it is vital to fully understand the needs of the functions inside the building as they will drive the best outcome. In a hotel the room width is the key dimension. The structural grid parallel to the facade will probably be one room wide between walls (only occasionally are two chosen for some specific reason). Specific uses and markets will have their own requirements and standards. The London office market for instance requires everything to be planned on a 1.5 m grid. The distance from glass to core wall is preferred to be not more than 13.5 m – from glass to glass is 18 m. The first column in from the facades should be more than 4.5 m away so that perimeter offices can be freely planned, and so on. In Germany the preferences for an office floor plate are completely different. The structural scheme must do its best to work within the appropriate functional rules.

Remember when discussing column sizes with the architect that they will take any dimensions you give them as external values of the finishes needed. They will be making promises to the client on the useful net to gross area ratios they are delivering and planning minimum escape widths alongside the elements, so these dimensions are important.

The finished dimensions on steel columns in particular need to be carefully considered. Firstly remember that the UK's Universal Column (UC) designations refer to the smallest section in the range. For instance, a 305 × 305 × 283 UC is actually 365.3 mm deep by 322.2 mm wide, give or take a millimetre or so for rolling tolerance. If you then have, say, a 25 mm splice plate on each flange this adds 50 mm to the depth, plus the bolts, which will probably have their threads projecting outwards as that allows easier access for the gun to do the nut up. Say another 25 mm on both sides. If the architect insists that the clad outer surface is exactly on grid you have to allow the column in a tall building to be out of position by up to 50 mm in every direction. Finally add on 40 mm all round for a skin of plasterboard.

So now we have talked up our 305 × 305 × 283 UC to a massive 645 mm by 502 mm. This can cause severe distress to an architect who has heard you say 305 by 305 months earlier. There are ways you can reduce the above numbers. We asked the contractor to have the bolt heads on the outside on one project I worked on for instance. This was not popular with him but increased the client's lettable space by 50 mm per column which soon adds up as significant rent gained in central London. Smaller column casings on floors without splices added more as did allowing the casing to be set out from the final position of the steel column rather than the grid. Similar issues need to be addressed for concrete columns and agreed with the architect before they become a bad surprise later.

It is also worth considering the impact that openings for service risers have in the building both now and in the future. The largest risers are for the ducts that move air between floors and these will normally need to turn out horizontally into the ceiling space, just below the slab of the floor above.

It is often the practice to position these risers inside the structural core. However, this means that the likely downstand beam or wall needed there will be a blockage to the air ducts turning out into the ceiling. Often the depth of ceiling void required, hence the floor to floor height, and hence the height of the building, is determined by this type of pinch point for the structure and services. Often risers are positioned outside the structural core, in effect creating a larger 'non-structural' core with easier access beneath the floor plate.

Future flexibility can be allowed for by designing 'knock out' panels alongside the core that can be opened up for new data risers or other purposes. The engineer should take care that opening these up in the future use will not cut the stability system off from the floor plate and that sufficient load paths remain to get force from the floor plate into the stability system.

7.5 Achieving the right structural section

7.5.1 The building as a whole

The structural engineer needs to think about the section of the building in two ways. On a macro level the arrangement of functions on various floors needs to be considered. On a micro level the arrangement of each floor's structure within the other floor systems needs to be coordinated.

A key first activity for the building's architect is to agree with the client where the building's different functions need to go within it. Where there is a mix of space types their relative locations can have major implications for the structural engineer and particularly in section if they are stacked vertically. The mix of long- and short-span spaces and the position of functions with sensitive movement or vibration requirements need to be understood. Often the structural issues arising can be avoided by reorganising the building planning at an early design phase, but sometimes there are other design issues, such as restricted sites and required adjacencies, that will constrain the overall design.

By way of an example, consider hotels, as shown in **Figure 7.5**. The majority of the superstructure contains bedrooms which have a regular, short-span rhythm of floor heights, walls, service risers and corridors, easily allowing close-spaced vertical structure. However, hotels also need open function and dining spaces. Thinking 'mono-disciplinarily', structurally it would be logical to place these at the top of the building to avoid the columns and risers from the bedrooms needing to be supported over them by transfer structures. However, function rooms and restaurants contain large numbers of people who need access and fire escape provisions. If placed at the top of the building the large vertical circulation required would impact the design through its whole height. As a result, these spaces are better placed close to the ground. If space allows, they can be placed outside the bedroom block's footprint, but that requires a sufficiently large area, which is not often the case for urban sites.

There is also a trend worldwide to place hotels at the top of tall multiple-use towers. The views from the rooms are the best, the prestige the highest and hence the price that can be charged increases. Physically the number of lifts to serve a hotel is much lower than the number to serve office buildings, where everyone has to go out to lunch during the same hour. Placing hotels above offices reduces the size of the lift core over much of the height of the building and hence increases the amount of usable space.

This is efficient for the overall design but conflicts with a key structural concern. Humans are most sensitive to the sway of building when lying down and at rest as they often are in hotels. The allowable accelerations from building sway are much more restrictive for a hotel than an office. Thinking only structurally, the top of a tower is the worst place for a hotel, but increasingly they are put there. Thus, the desirable solution for the project as a whole may require the engineer to design a much stiffer lateral stability system than the structural optimum.

However, a key issue to strive for is having as many columns as possible to run through the building from top to bottom. Indeed, many seismic codes preclude discontinuous columns without stringent studies and penalising precautions. A slight deviation in the position of a column between floors can cause very high local forces requiring careful detailing and restraint forces from other members. A more significant displacement, or indeed the deletion of a supporting column, will require a transfer structure which is usually a member significantly different from those around it that requires special design and construction consideration.

Changes in building function between various levels and in particular the requirements of vehicle circulation at the base of

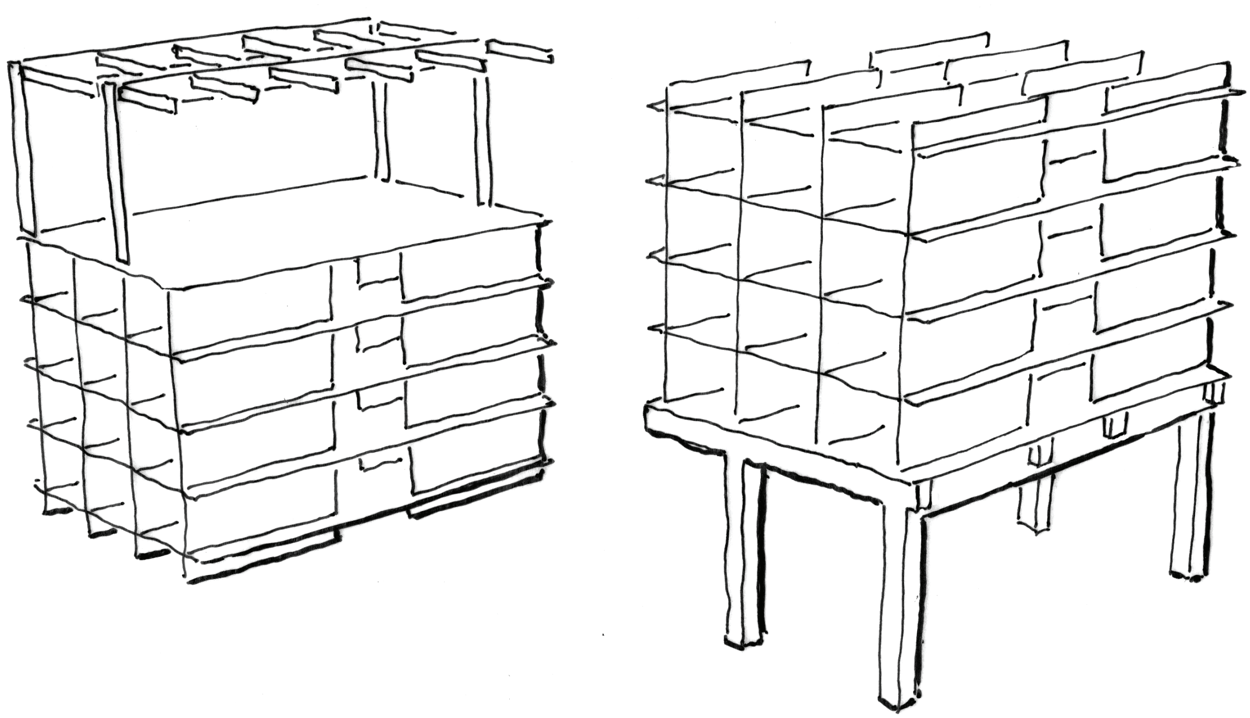

Figure 7.5 The structural engineer's and architect's preferred mix of hotel space

the building often means transfer structures are needed and in some markets whole levels of podium transfer beams are used above the traffic. In Hong Kong, for instance, there will often be a significant transfer podium level above a ground floor Passenger Transfer Interchange of bus and taxi lanes. Beneath this there may well be another level of transfers to adjust to the tighter grid of a parking basement or rail station beneath. The structurally easy life, with all columns running from top to bottom in all locations, is not an achievable ideal. Just don't stop striving for it or you will end up with a much more complex reality than you would otherwise get.

7.5.2 The multi-disciplinary floor zone

We now switch from the 'macro' issue of the building as a whole to the 'micro' issue of coordinating and minimising the multi-disciplinary floor zone. A simplified zone for a steel framed office building is shown in **Figure 7.6**. The structural engineer's primary consideration is for the structural elements – the slab and the beams, but they only form part of the total.

Starting from the top, the slab will support finishes. In this case it is a raised floor for cabling, but it could be screed and carpet in residential buildings or a roofing build-up at the top of the building. Then comes the slab and beneath that the beams. Steel beams require fire protection and a vertical dimension for this needs to be allowed for. However, remember that in the real world the beams and slab will not be at the right level or be the right size, within the limits allowed for in the specifications. Thus, a tolerance needs to be included.

The ceiling beneath will eventually be installed flat, often now very accurately with a laser level and the same is likely to be true for the raised floor above. This will occur after the structure is free-spanning and dead load deflection has taken place. Thus, an allowance needs to be made for this deflection. A residual gap is then needed beneath the beams to allow air ducts and cable trays to pass underneath. Always remember that the building services engineer will tell you the internal dimension of ducts, the one they have calculated for the air, and not the overall external dimension including insulation which you need to know. Finally, beneath all this is a suspended ceiling, but space is needed not just for the tiles but for the light fixings that can project upwards by over 100 mm at many positions.

The situation described above is 'uncoordinated', with every item having its own vertical zone and not sharing with others.

The floor to ceiling headroom required will be strictly defined by the client's brief or market requirements. In the central London office market for instance it is likely that anything less than 2.75 m will be unlettable, and there won't be any extra rent for more. Depending on structural spans and other systems, an uncoordinated floor zone can push floor to floor heights significantly past 4.25 m and beyond.

Excessive floor to floor height adds considerably to the cost of the building. All vertical elements, including cladding, risers,

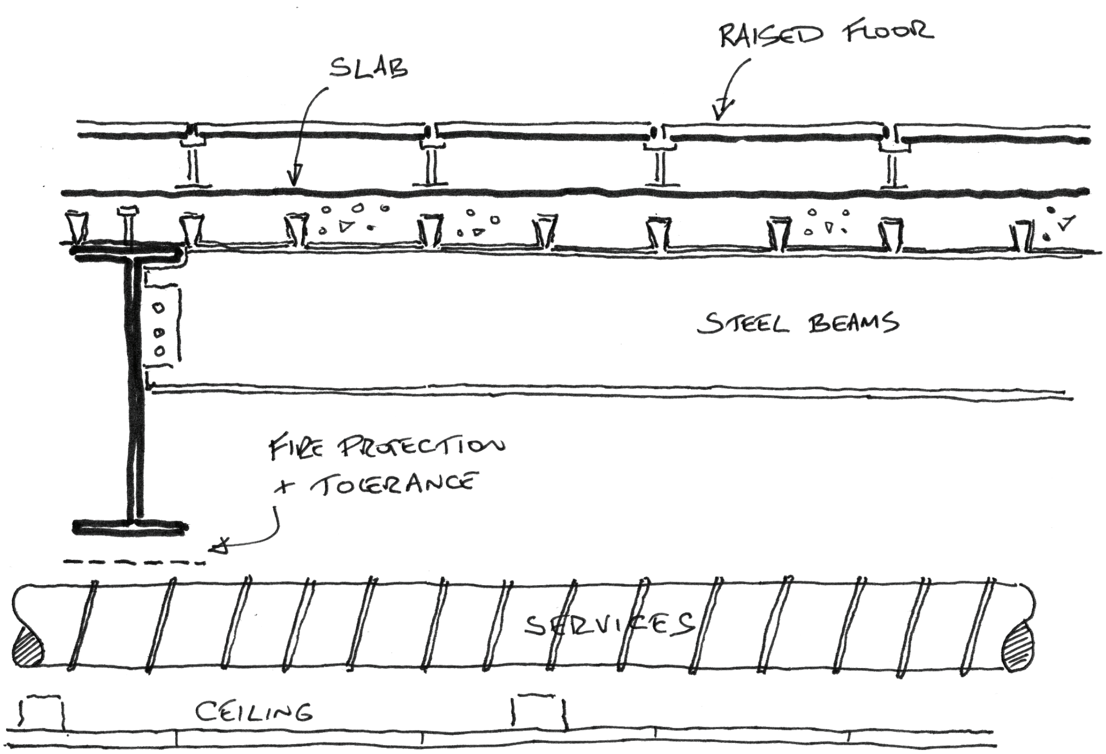

Figure 7.6 Typical uncoordinated multi-disciplinary floor zone for an office

stairs and internal walls, increase in quantity. Additionally the internal volume of the building increases which requires more plant to condition it. There are big advantages for the project's budget in squeezing the floor zone thinner.

In some markets, the size of building the client can construct is set by a cap on building height, not by a limit on floor area. Staying in the central London market, building heights are set by 'St Paul's heights', the requirement that the dome must be visible from key points around the City. This places a non-negotiable height limit across sites. I have carried out detailed multi-disciplinary coordination early in concept to squeeze millimetres out of the floor zone and enable an extra floor across a site. The extra future rent for the client fully justifies this early extra effort.

The first obvious way to reduce the depth of the floor zone is to minimise the depth of the key elements. The building services engineers make air ducts wide and shallow, overcoming the friction inefficiencies caused. Similarly the structural engineer adopts minimum depth solutions rather than more efficient minimum weight sizes.

To further reduce the depth of the floor zone the structural and building services engineers need to share each other's space. There are three strategies of increasing sophistication that can be considered.

For all structures other than concrete flat slabs there will be areas between the downstand beams where the headroom goes up to the soffit of the slab. If there are large items of plant within the ceiling such as variable air volume (VAV) boxes these can be positioned in these vaults to avoid them driving the required depth up.

As the next step some building services can be taken through holes in the structural beams. Often sprinkler pipes are the first step towards this, avoiding problems clashing with other services. It can be more difficult to take anything more than a cable tray through a concrete beam, but often significant holes are carved through rolled steel beams or plate girders for air ducts. These holes are often placed at mid-height of the beams and in the middle third of the span, avoiding higher shear at the end, and will likely need to have stiffeners. Detailed calculations are required for these holes. In recent years automated fabrication techniques have allowed some manufacturers to competitively offer fabricated beams that effectively fill the whole depth of the floor void but have large circular openings through nearly all their length.

A final strategy that can be used is to taper the depth of the beam towards its supports. This is normally used in steel frames but can also be useful for concrete schemes. This allows large ducts to pass through increased depth zones close to columns, reducing the overall depth.

These holes and tapers all add complexity and cost to the structure. However, for a building driven by external constraints they can result in solutions that create significant value for a client. If an extra floor can be built across some or all of the site that is a step-change in future income for a landlord!

Some of the ways that structure and service zones can be integrated are shown in **Figure 7.7**.

The layout of the columns, discussed in the previous section, can also be an important factor driving the section and the levels up the building. As mentioned above, for all solutions other than a concrete flat slab there will be structural downstands of

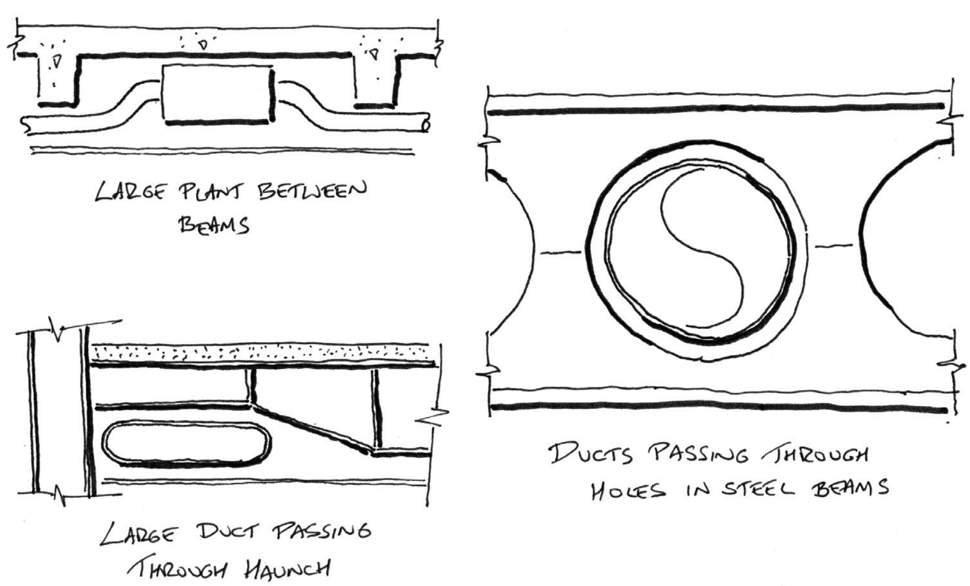

Figure 7.7 Ways to integrate structure and services

either steel or concrete ribs or beams. Within a bay the usual configuration is for the slab to span between secondary beams which in turn span the longer direction of the rectangular bay defined by four columns. This will tend to minimise the depth of the primary beam that supports the secondaries and hence minimise the overall depth of the structural zone.

However, this will not necessarily allow the shallowest overall floor zone. If it is known that the largest service runs are in one particular direction the directions of span can be swapped so that the secondaries span the shorter dimension and become shallower as a result. The large services running parallel to the primary beams in this rectangular bay can move upwards to the underside of secondaries, reducing the overall depth of the floor zone. This configuration of rectangular bay can be useful in reducing floor to floor heights in, for instance, airports, where the large baggage handling conveyors in ceilings might otherwise pump up the building's height (see **Figure 7.8**).

One further integration of structure and services is through the creation of beam-free zones (see **Figure 7.9**). By positioning columns on either side of the major service runs, the secondary beams can span straight to them, with only slab in between the lines of beams. Major duct runs can be hard up against the slab with only smaller distribution ducts needing to pass beneath the beams, dramatically reducing the multi-disciplinary floor zone required.

7.6 Accommodating other components and issues

The structural engineer will often need to get involved in 'non-structural' design issues of importance to the wider team: the architect, building services engineer or contractor. It is important to plan ahead and agree the scope of the engineer's input to these shared concerns to allow efficient progress for the team as a whole.

These issues will not need to be fully described on the engineer's drawings. The information may be passed to others for use on their drawings or coordinated allowances made for connections and loads from building systems.

Smooth design progress for the team as a whole is ensured by agreeing the scope of 'who does what and by when' for these issues across the disciplines as early as possible. The multi-disciplinary team functions best when all team members are looking out for each other's interests and anticipating problems, opportunities and needs. Generally, the more structural

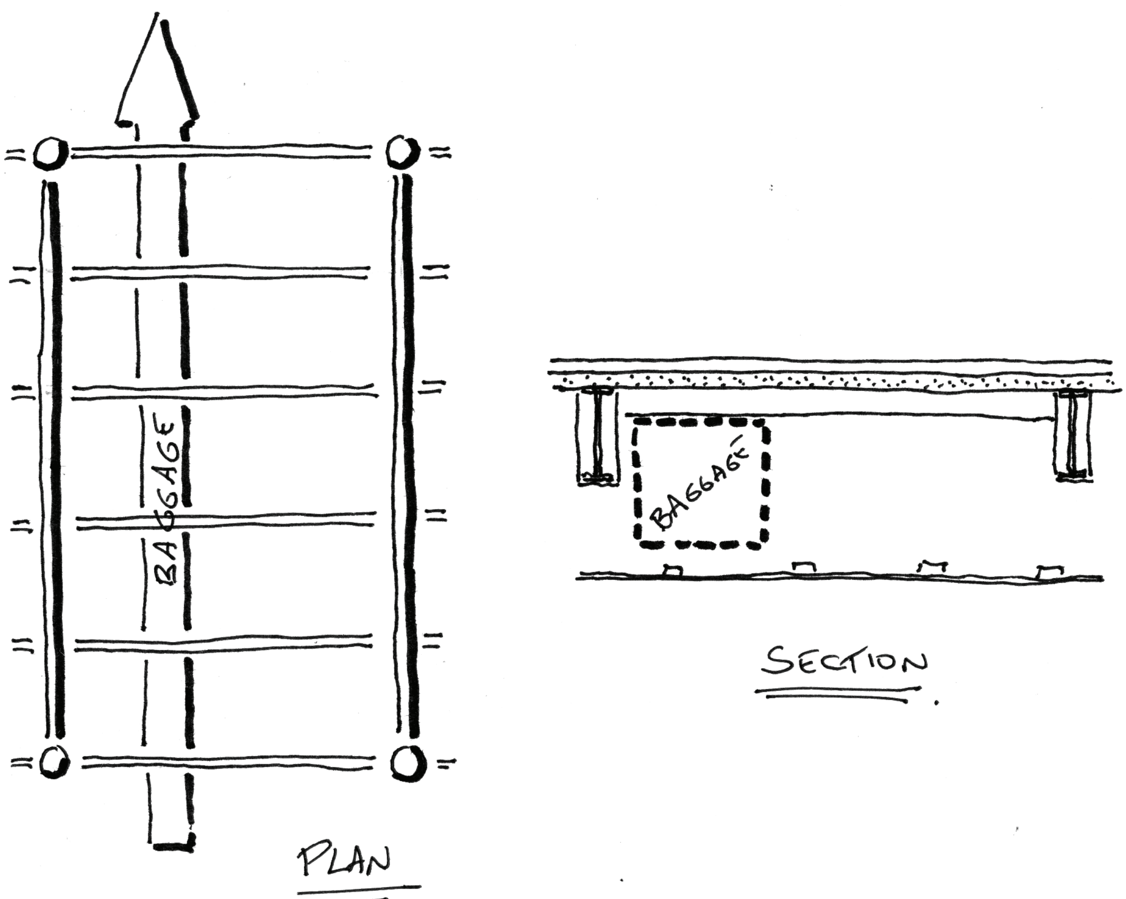

Figure 7.8 A conceptual bay arrangement at an airport

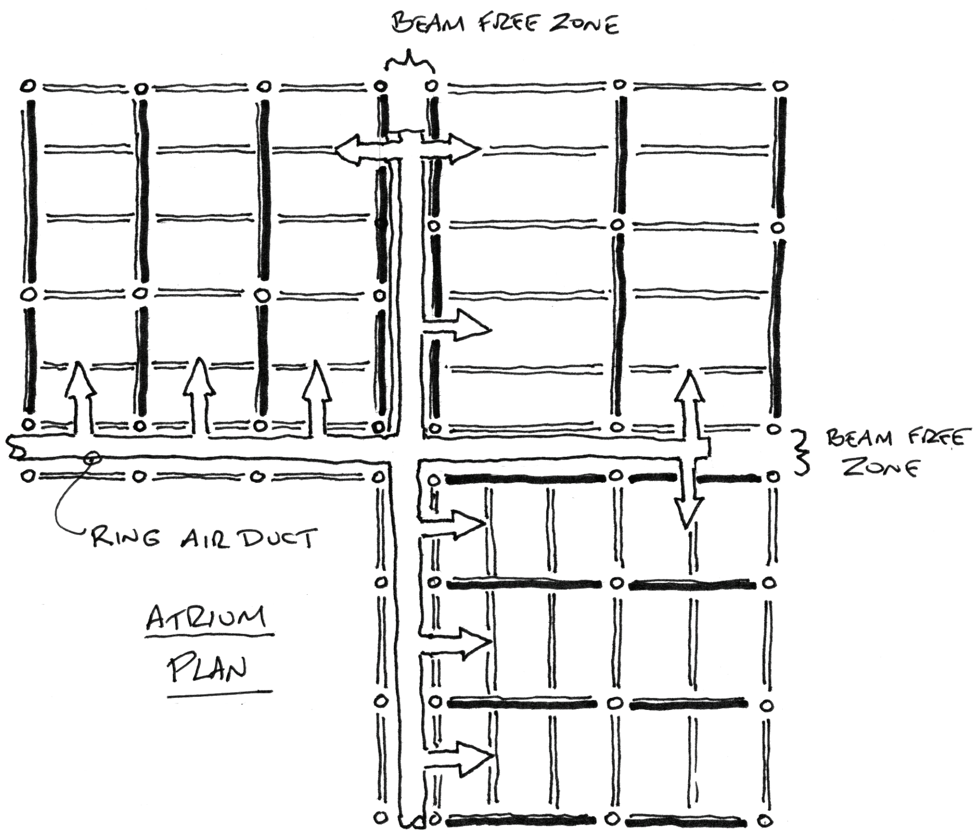

Figure 7.9 Beam free zones at Wimbledon Bridge House

engineers know and understand about the workings of the building as a whole, the more they are able to contribute to the success of the project as a whole.

Some of the issues that the structural engineer may need to contribute to include:

- partitions and cladding;
- plant and openings for services;
- fire and corrosion protection;
- vibrational and acoustic behaviour;
- stairs;
- cold-bridging and waterproofing;
- sustainability and buildability.

If the internal partitions are 'structural' – load-bearing concrete or masonry – these are part of the building's vertical load path and their design should be 'owned' by the structural engineer and fully shown and dimensioned on the structural drawings.

However, not every wall is likely to be structural and definitely not every return around a door frame will be of importance. If the engineer shows every architect's detail they are likely to later be committed to multiple reissues of drawings as the interior layouts are finessed.

It will save the engineer time and pain if a series of typical structural details for the non-structural walls are established that show reinforcement and lintels around doors and openings, posts to stabilise larger panels and rules defined that allow the architect to divide up the walls with movement joints to control cracking.

Load-bearing walls, by their nature, need to be in contact with the structure above. Conversely, it is essential for non-structural walls, which can include lightweight partitions, to have sufficient movement allowance above and to the side to ensure that they do not attempt to carry loads they are not designed for. The structural engineer must establish a realistic movement allowance at the wall's head which can accommodate the structure's vertical and horizontal deflections from imposed loads and whatever proportion of dead load movement will occur after construction of the wall. There can be significant advantages and savings if the structural engineer can set this value to a realistic minimum as smaller figures can simplify – and cheapen – the fire, acoustic and restraint details at the head and sides of the walls. It is also important to limit the deflection of all slabs after construction of the supporting walls or partitions to values that will not cause cracking in them.

As mentioned earlier in this chapter the loads assumed to act on the structure should be sensible minimums. Some teams use loads in excess of the planned imposed load to represent the weight of masonry partitions, allowing future flexibility for their positioning. This is an easy decision for the structural engineer to make early in the design process and enables progression without the need to coordinate, but the wastefulness in money and resources must be recognised. It is important that the engineer explores with the client and team whether lighter weight partition options might be suitable and also tries to design for as many firmly fixed locations of walls as possible, eliminating these inefficiencies.

If the external walls of the building are of masonry, either load-bearing or non-load-bearing, the issues to be addressed are the same as discussed above, with the additional important issue that the engineer must define the wind loads acting and design and detail the wind posts that will be required.

If instead the building has a curtain walling system it is unlikely that the structural engineer will be responsible for its design, this instead being carried out by a specialist subcontractor. However, it is important that the engineer understands how the cladding works and its support requirements, as these must correspond with the performance they are delivering for the overall deflection of the building, the deflection of supporting edge beams after cladding installation and the setting out of the slab edges. The engineer should press the architect to fix the location and details of cladding connection brackets as these can impose local moments on members, requiring additional concrete reinforcement or bracing of steel members as well as cast-in, welded or drilled connections ready to receive the brackets.

As is the case for partitions and cladding, the structural engineer should define rules that allow the building services design and the structural openings it requires to be developed and constructed without having to coordinate and draw every location. The key issue is to determine when a hole or load is 'structurally significant'. This may include, for instance, holes through slabs that are less than 300 mm in diameter, not near column heads in flat slabs, or for beams, 100 mm diameter holes at the mid-height of beams in the middle third of the span. The engineer should be satisfied that the structure can accommodate whatever their definition of structurally insignificant is when in the worst location or combination and ensure their design thus allows a useful degree of 'general flexibility'.

Other 'significant' openings should be coordinated individually with the structure and shown on the structural drawings. The type of structural form adopted will influence the level of attention to detail required from the structural team. The greater the emphasis on off-site construction, for instance in precast concrete or steel members, the greater the need for all holes to be defined.

Larger items of plant are often installed on plinths, and the additional load from these can add significantly to the loads in a plant room. Any tanks, pipes or plant with water or oil can be particularly significant for loads. In the United Kingdom, 7.5 kPa is often used for plant rooms and this normally easily accommodates large air handling units and their plinths – indeed I have seen these loads talked down to 4.5 kPa by careful study. However, large water-filled cooling towers or chillers, or battery racks for uninterruptible power supplies can rapidly head up to 20 kPa over their footprints.

Loads in the room as a whole can sometimes be reduced by consideration of the area around the plant being more lightly loaded, allowing only for servicing. Note, however, that large pieces of plant do not last forever and building services engineers should define access routes and openings for these. The structural engineer may need to allow for higher loads along these routes, crane-rails above if needed and corridor widths and column spacings need to be carefully checked.

Pipes in vertical risers are often not vertically supported floor by floor, allowing expansion to occur, and thus the whole weight of pipe and water may need to be supported at the bottom. This can be a very significant load in tall buildings. In pressurised systems there may be high local thrust loads when pipes change direction. These loads need to be understood and supported, which can cause complications on steel frames in particular.

Normally suspended services can be seen as a blanket load applied to the underside of the slab, but rules should be set for the building services team on what and how things can be freely suspended so that they understand what items are exceptional and thus should be flagged in detail to the structural team for review.

It is important that the lines of responsibility for the specification of fire and corrosion protection are defined between the structural engineer and architect. In buildings it is normally the architect who will set the fire period requirements for the various components. If the structure is of reinforced concrete the engineer is then responsible for defining appropriate concrete cover to rebar. If the structure is of steel it is likely that the details and specification of the board, blanket or spray protections will be provided by the architect. Corrosion protection systems should normally be defined by the architect but unfortunately some only focus on the colour of the finished coat. It must always be remembered that for steelwork 'paint' is actually a system of a series of layers and processes, starting at blast cleaning. Since this work will be done by a contractor carrying out a 'structural' package it is often the structural engineer who will need to define the requirements.

When considering the structure's vibrational and acoustic behaviour the structural engineer needs to understand the sensitivity of the building use and the susceptibility of the structure. The architect should be encouraged to sidestep problems by separating incompatible functions. I know from first hand that it is almost impossible to keep the noise of a basketball bouncing on a rooftop court out of classrooms beneath, despite costly measures. It would have been better to avoid the problem by moving the court elsewhere as it is always cheaper to reduce or eliminate at the source rather than try and mitigate for the receivers. Rhythmic or noisy activities or impacts, and long span or lightweight structures should alert the structural engineer to potential problems, and where suitable performance criteria are not available from standard sources, specialist advice should be engaged.

Design of stairs, whilst not often affecting the design of the main structure greatly, normally takes a lot longer to finalise and coordinate than the structural engineer anticipates. Normally, after early simplistic design, the architect will do their final coordination after the initial priorities, such as cladding and other major packages, are resolved. On a fast-track project, the structural engineer should recognise the design problems this will cause as they finalise the structural packages, particularly if precast concrete or steel staircases will be erected with the frame to give construction access.

Fully detailing a staircase often involves more drawings, and at a larger scale, than is initially planned for. In particular, note that the number of drawings detailing a steel stair flight can be three times the number for a similar reinforced concrete stair. The more ‘architectural’ emphasis, the more attention to detail will be required as aspects of the structure become the ‘finish’ of the building.

The architect may need assistance with sizing typical handrail posts and connections but the structural engineer should avoid coordinating these beyond general advice as there is a world of pain and regulations that need architectural ownership.

Although not the structural engineer’s problem, they should be aware of the role that the structure plays in causing cold bridges and compromising the performance of the building’s waterproofing. Any structure that projects through the insulated skin should alert the team to the possibility of a cold bridge and thermal lagging over a length or specialist structural details may be required. The structural engineer can usefully alert the architect to problem areas for the waterproofing, in particular places where there are local differential movements that could rip membranes or reverse drainage falls.

For the structural engineer on ‘conventional’ buildings sustainability issues are only just coming into focus. Design parameters are beginning to be agreed, supply chains starting to supply data and definitive advice slowly becoming available. This developing subject is beyond the discussion here but as the operational energy over the building’s life drops through efficiency gains in equipment, envelope and usage in the future the emphasis will move on to the building’s embodied energy. The structural engineer has a key role in specifying low-carbon materials such as pulverised fuel ash (PFA) and ground granulated blast furnace slag (GGBS) in concrete and considering future maintenance and reuse of buildings and elements of their structures. In appropriate climates and locations the structural engineer can work with the architect and building services engineer to eliminate finishes, expose structure and use the thermal mass of the building to control the environment within the building.

It is worth noting that the best way to reduce the embodied energy of a structure is through minimising the loads it is designed for, choosing the most appropriate structural scheme and through the specification of its materials. Often, discussion focuses on optimising member design but only once these earlier items have been correctly achieved. This final step has less influence than the others.

Most of this chapter has focused on the structural engineer’s work with other designers to fulfil the client’s brief. However, huge value can be obtained by their working with a contractor already appointed or by correctly anticipating the preferences of a future contractor. Designing a solution that suits the site location and market the building is being built in, and that works within the limitations that the site places on access and plant, especially around roads and railways, can result in a better, cheaper building. Repetition of building elements and measures that allow off-site prefabrication all contribute to increasing this ‘buildability’ and speed of construction on site.

7.7 Tall buildings

As mentioned in the introduction to this chapter, as buildings become taller the vertical circulation, service risers and horizontal stability structures increase in importance for the design and begin to merit extra consideration both in detail and for their overall impact on the building as a whole.

When is a building ‘tall’? The perception of this is often influenced by the local market. In London a building over 20 storeys (approaching 100 m) might be considered tall, whilst in central Beijing it would be shorter than usual current practice. However, comparison with local markets is not useful in this discussion. It is better to consider the British Council for Offices’ definition that ‘a tall building is not a low building that is vertically extruded, but one that is differently designed’.

As this implies, there is a continuum as the design becomes taller, with various issues reaching thresholds requiring solutions technically distinct from lower-rise buildings. Often these issues require greater study and optioneering by the team as a whole at an early stage in the design process, which should be commercially feasible for the designers within the larger fees resulting from major structures.

Structurally, the key early consideration is establishing the appropriate strategy for resisting the horizontal forces acting on the building. For the lowest rise building, these forces might be carried by moment connections between the vertical and horizontal elements or by discrete walls or bays of bracing. Such strategies will have little or no impact on the planning of the building by the other disciplines.

However, as the number of storeys increases, solutions with a greater impact on the holistic design are needed – first with walls and bracing being connected into larger cores, then solutions using the columns in the building’s skin to create perimeter tubes of increasing density and sophistication.

As buildings move above forty storeys, the core and perimeter structure often need to be connected at intervals up the building height by outriggers that couple their performance and stiffen the building. These outriggers have a major impact on the spaces they pass across and are often two storeys deep.

Conveniently the building service engineers are often looking to have two storey deep plant rooms at intervals of around 20 to 25 storeys up the building to reduce the vertical distance the risers serve. The usual solution is for the structural and services engineers to coordinate these zones to serve both their purposes. Taranath (1997) gives an excellent overview of these developing issues heading up to 120 storeys and beyond.

As discussed earlier in Section 7.3, since structures are designed to support loads it is vital to accurately identify what the design loads should be in order to achieve an efficient structure. As buildings become taller the wind becomes the source of the dominant loads that will drive the sizing of the structural elements. Even in seismic regions it will often be the wind that can drive the design of the tallest buildings – although seismic considerations will remain vital.

If the height of the building is beyond the strict remit of the local codes (for instance, in the case of BS5950 the code was limited to 300 m) it is evident that specific study and advice from experts is needed to establish wind loads, probably with the use of wind tunnel tests. Even at lower heights it is often worthwhile engaging wind specialists at a very early stage. In dense urban centres or for buildings of complex shape they can advise and sometimes significantly bring down the loads for which the structure must be designed.

The structural engineer will often be the focus of the design team's relationship with the wind specialist. However, it is not only the structure that can benefit from specialist advice and testing. Accurate wind pressures will be of great use to the cladding designers allowing them to design an efficient solution and reduce the cost of one of the most expensive items on the project – the building's skin. Also, the architect will need to know how the building changes the flow of wind around it on a daily basis – not just for the extreme events that interest the structural engineer. Changes to the building form can mitigate wind flows that can greatly affect the pedestrians around the building.

However, the structural engineer needs to be aware that there is a very tight window of opportunity in which to gain information useful for the structural design process. Often the building form will not be settled until towards the end of the conceptual design period. If the decision to undertake a wind tunnel test is not made at the earliest possible stage, it is likely that the results will not be available until scheme design is finalised, severely limiting the advantage of the results for the project. It is worthwhile persuading a client that testing is necessary right at the start of a project, and beginning conversations with the specialist as soon as possible after this is agreed.

For very tall buildings, the lateral stability system is not just judged on strength or deflection (overall or inter-storey). It is also important for the comfort of occupants to control the horizontal accelerations they experience – normally for those at the top of the building but sometimes also around mid-height if the second vibrational mode is significant. As discussed earlier in Section 7.5, this can be particularly important for hotels. Wind experts will be able to assist the structural engineer to understand the way the spectrum of force variations in the wind loads interacts with the vibrational periods of the building for both along- and cross-wind movements.

As the needs of the lateral stability systems increase with the building's height it becomes increasingly important that the chosen solution is coordinated within the design of all the disciplines. One key area needing attention will be the design of the core as many parties will be placing increased reliance on it:

- The vertical circulation specialists will need to achieve groups of elevators that can efficiently serve the building's accommodation within its overall lifting strategy.
- The architect needs to accommodate the staircases that escape and local circulation require plus the appropriate bathroom, storage and lobby areas.
- The building service engineer will need to balance having larger plant rooms in the core on each floor against having larger risers to central plant within the overall servicing strategy.
- The structural engineer needs to establish the lines of walls and bracing needed to resist lateral forces as well as carrying the core's vertical loads at each floor.
- Finally the architect will need to squeeze all these elements into the tightest possible core configuration to maximise the 'net area to gross area' efficiency ratio on every floor.

When designing a tall building the team should hold regular multi-disciplinary workshops focusing on the core from an early stage in order to achieve an optimum solution. This extra effort is justified through the impact it has on every floor as it rises through the building and thus the economic competitiveness of the building.

In parallel with this emphasis on the lateral stability systems there is also great advantage to the project if extra effort is made to optimise the design of the floor systems. Thinner and lighter becomes very desirable as the building gets taller. If the combined floor zone of structure and services can be reduced it is likely that the height and thus cost of the building can also come down or additional floors constructed, increasing client income. Reductions in the mass of the building will pay dividends through the reduction of vertical load on the columns and foundations as well as influencing its horizontal behaviour.

The columns and walls of tall buildings are often more highly stressed than those of low-rise structures. When considered over the height of the building this means that axial shortening, both from elastic and long-term creep, can be significant. If some vertical elements across the floor plate are more lightly loaded than others relative vertical movements can build up through the building's height. This is sometimes significant around cores where elements can support only stairs or risers, but are close to others holding up large areas of floor plate. Careful consideration is needed to prevent damage to their connecting elements and finishes.

7.8 Summary

A multi-storey building is a complex three-dimensional multi-disciplinary object. Development of the optimal structural solution requires the orderly investigation of concepts, the selection of the appropriate scheme and development of the required details for construction. It is vital to understand the client's brief, the mix of spaces, the requirements of the other design disciplines and to work towards a fully coordinated design that has appropriate future flexibility without overdesign.

Early communication of the key features and requirements of the structure will allow other disciplines to understand and coordinate with it appropriately. Sketches, drawings and visualisations are all tools that help achieve an agreed multi-disciplinary scheme, enabling the development of later detail to proceed efficiently and with confidence.

Within the multi-disciplinary systems of the building it is important that there is a clear and robust structural diagram. Gravity and horizontal forces should flow to the ground along direct and understood paths.

The spacing of columns and the positions of cores and movement joints are key decisions to be agreed. Both architects and building service engineers place great emphasis on the cores due to vertical people circulation and service risers. If these are also the structural elements that provide horizontal stability through bracing or walls they become a vital focus for coordination within the wider team. The column spacing is naturally the primary influence on the structural sizes required for the beams and slabs. As these will probably represent the majority of structural material and cost in the building, the engineer must work to achieve an optimal solution.

The floor systems are often key drivers for the height of the building. The zone they will occupy within the multi-disciplinary section through the building needs to be understood and coordinated at an early design stage to allow other disciplines to proceed with confidence. Pinch-points, where all disciplines and systems come together, often occur at cores and facades and along the primary service route. These areas should be studied in detail early in the design process. If columns are interrupted through the height of the building the local disruption to the rhythm of building components caused by the transfer structures needs to be appreciated.

The structural engineer's input extends beyond just the primary structure. There are a variety of components and issues where it is important to understand whether they are the structural engineer's, architect's or others' responsibility. Cladding systems, finishes and partitions are natural interfaces with the architect, and support of major plant and suspended services with the building services team. Primary responsibility for fire and corrosion protection, cold bridging and waterproofing will probably lie with the architect, but the engineer has a key role to play in their success.

Last, but not least, and with increasing emphasis, the chosen structure needs to be optimised to reduce the overall environmental impact of the building over its life-cycle. As improvements to the operational energy efficiency of our buildings continue we will increasingly require the embodied carbon of our structures to be assessed and minimised – an emerging challenge for the future.

7.9 Conclusions

This chapter began by noting that the success of multi-storey buildings must be judged across all design disciplines and be seen through the client's and user's eyes. All the requirements subsequently discussed are best achieved by teams where the structural engineer is a key participant, striving for the minimum loads, efficient structural layout and the appropriate structural system, working within and contributing to the success of a multi-disciplinary whole.

7.10 Note

[1] Please note the artwork for these chapters feature the author's hand drawings as would be done in practice during the design stage.

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CONTENTS

Chapter 8

Typical design considerations for generic building types

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Engineering graduates may feel that they leave university and enter industry with the knowledge that 'they can design a structure'. However, the specific knowledge they may well lack will be the subtle differences or indeed focuses of consideration that will be presented to them at the outset of design, in seeking to fulfil an individual brief set by a client for a building with a defined end-use. A client expects, and has the right to expect, that the engineer they employ will know what to deliver for their particular building use. The art of a good engineer is to be in a position to know what the structure for an end-use should deliver in terms of loading allowances, column grid, performance, floor heights and the like. Each end-use will have aspects of these which need different consideration or focus on one more than any other. This chapter seeks to inform engineers about to consider a structure for a particular end-use of the areas of design that they should specifically consider in order to deliver an appropriate brief for a client.

8.1 Introduction

One of the most important factors when considering a structure which is to be designed is its intended use. There can be no 'one size fits all' solution since the whole structural solution must suit a variety of needs, most of which will be determined by its use. However, few solutions can, or should, be so narrowly focused such that they only suit a defined need at the time of design or construction. Engineers considering today's needs may be required to consider a reasonable and economic level of flexibility to ensure that the structure is readily and economically adaptable and able to accommodate subtle changes to the use which might be reasonably predicted. This aspect will particularly apply to considerations of floor loading and use or placement of any load-bearing or shear walls or bracing elements.

One could say 'a structure is just a structure': it does not know where it is or what it is used for. Nevertheless, the arrangements and design parameters need to be subtly different so that the designed structure will suit the unique end-users' needs and ultimately the client's brief. Every new building should be considered as a 'blank canvas' at the time of inception and the engineer's role is to determine the most suitable and economic structural solution within the defined envelope (both horizontal and vertical) that will economically deliver the brief and suit the ultimate needs for its everyday use.

An engineer, in determining and advising on the structural design brief for a building defined for a particular end-use, therefore needs to understand what the key consideration criteria will be such that the building is able to successfully serve that use (see **Figure 8.1**). At the same time, the design will need to fall within the bounds of available budget and be considered to be economic and offer best value for money. The design must also fully consider the constraints of the site to deliver the brief and not attempt to deliver something that might rely on aspects not within the client's control such as boundary junctions, party walls or effect on any adjoining third party especially from construction.

It is intended therefore that this chapter will serve to inform engineers, considering the early stage brief and scheme development of the structure for a building, about the differing and pertinent aspects which should be thought about at these initial design stages for a client.

Pertinent design considerations for a variety of building types which an engineer is most likely to encounter are discussed in the following sections. All buildings will generically share requirements for three key 'ingredients' to be considered, namely, loading requirements, column grid and storey height.

At the briefing stage, through liaison with the client or their advisers, the engineer should also become familiar themselves with the 'estates' and associated legal and land ownership issues which may influence and sometimes govern the type of structural solution proposed. These issues are all too readily forgotten or misunderstood as the briefing and initial design process unfolds and they can return much later to seriously affect the project.

8.2 Hospitals

Hospitals present unique structural challenges compared to many other building uses. Such buildings will have many varying internal uses which will place particular emphasis on more specific and detailed consideration of noise (both air- and structure-borne) and vibration sensitivity. They will invariably be more compartmented than most buildings and they will be heavily serviced throughout the space, whatever the particular medical or clerical use. Furthermore, they will need to more readily adaptable to future changes in clinical use since equipment, medical treatments and operating techniques advance and change more rapidly than many other end-uses. These changes may give rise to potential variation to the building arrangement or application of loading over time.

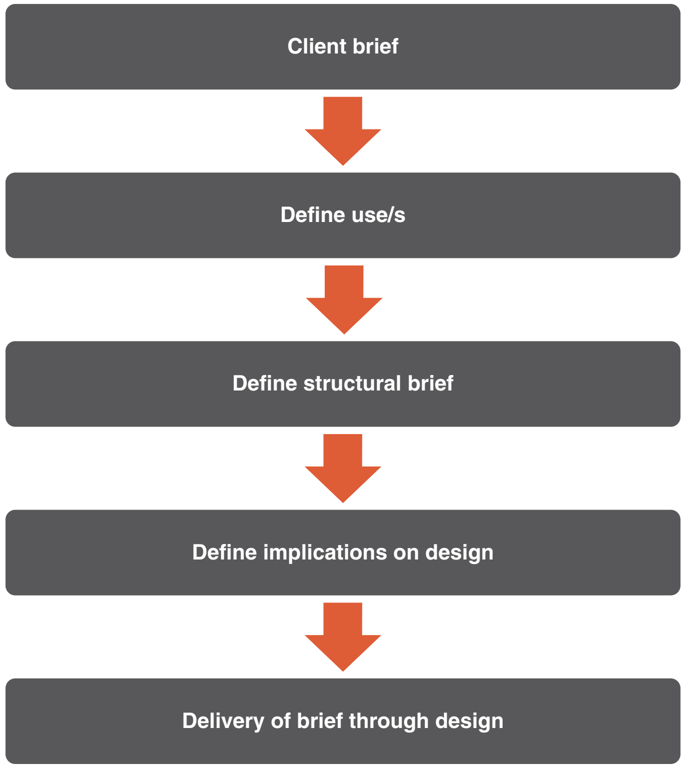

Figure 8.1 Consideration process

It is therefore important to consider that during the lifetime of any hospital building, the initial defined use of the space is likely to need to change many times and so the ability for that space to be flexible will be one of the key drivers for structural solutions for the frame. This may drive the need for an economic long (clear) span structure where the small extra cost associated with such a solution is typically more than offset by the benefit of improved future flexibility.

The placement of columns within any floorplate is an important consideration and careful thought must be given to how this might influence the arrangement of internal division walls or compartmentalisation of space which could vary greatly depending upon the use, not just at the outset but in the future.

It is recognised in the medical care field and particularly in the patient healing and recuperation process that natural daylight plays a significant role. Buildings designed for ward use should therefore be shallow in depth so that daylight can penetrate fully into that space without, for example, the encumbrance of perimeter downstands restricting the height of windows, or structure which might hinder the daylight pathway (see **Figure 8.2**).

To achieve the greatest flexibility, consideration should therefore firstly be given to the potential for achieving economic clear-span space. Spans of up to 15 m may be economically achieved using steel beams supporting a concrete floor system on a panel or column bay width of say 3 m or 6 m. Such a solution could be combined with web openings in the beams to facilitate services coordination.

In order to provide a flexible design to facilitate potential occupancy or use changes in the future, generalised live load categories should be applied to large areas, preferably one category to any one floor. For example, designing for a reduced load (as required by code or use) over a small floor area of a whole floor requiring a higher load would not be practical or realistically economic. Such a solution may also limit future flexibility to change of use.

In hospital buildings, of key importance is consideration of the structure for limitation of vibration. It will be appreciated that hospital uses involving sensitive equipment and particularly operating theatres need to be functional within the whole context of a larger hospital environment. However, such sensitive uses may require limitations to be applied on the performance of the structure as a whole such that there would be no residual effect on these functions of use. For example, due to slab continuity, panels adjacent to any areas specified for sensitive use would duly have to be considered for the effect they may have, particularly for footfall type vibration causes.

In the UK the strict requirements of the National Health Service (NHS) are defined in HTM 2045 (NHS 1996) which provides design criteria in support of BS6472-1:2008. The consideration of vibration, generated typically by footfall but also by the likes of dynamic plant and equipment, is important not

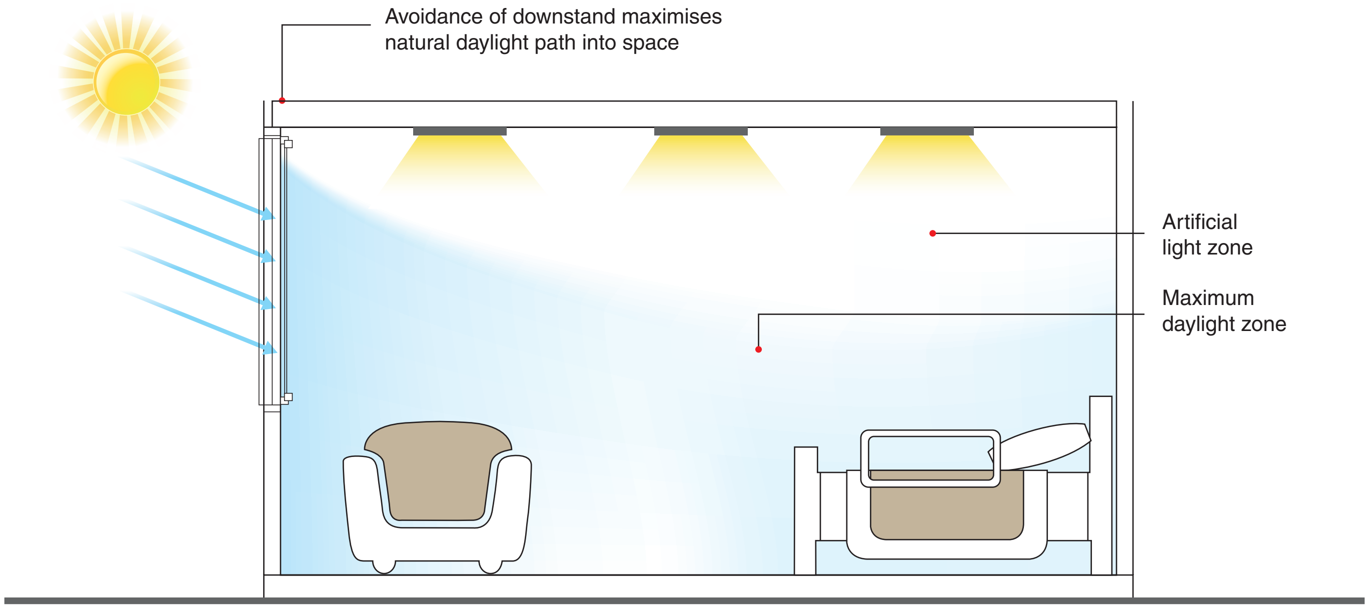

Figure 8.2 Hospital space design

only for sensitive uses but also to a lesser extent on wards where the night-time period will be more sensitive than the day use.

The precise manner in which a structure will perform under vibrational forces, particularly footfall, is complex to assess due to the many parameters which will affect this. These include span conditions, element fixities (slab and beams) deriving from continuity conditions and panel adjacencies, mass of elements, form and placement of load; essentially conditions which would otherwise serve to dampen vibration. It is therefore essential given the general sensitivity of use that a structure for a hospital is designed using simplified guidance that will serve to limit vibration. This is of particular relevance to steel framed structures which may have greater sensitivity to effect of vibration or 'bounce'.

The tendency for a structure to transmit vibration can be limited by the use and application in specific vibration design calculations of multiplying factors referred to as 'response factors' which in respect of hospitals are defined in NHS HTM 2045 (NHS, 1996). These factors are given in **Table 8.1**. These response factors define how a structure will behave relative to a 'base curve' and the multiplying factor which defines the acceptable limit of vibration behaviour is referred to as the 'response factor' which serves to limit the natural frequency of the designed element, typically a floor support beam.

Use of these factors in vibration design assessments should result in structure, particularly floor support beams, sufficiently stiff such that in normal use the effects of vibration would not be discernible or have a material effect on the required use.

The floor to ceiling height requirements in hospitals will depend on a number of factors which should be considered in the development of the overall design. These may include the specific needs of the use of the space, the nature of building services solution and floor finish requirements. The engineer may also need to consider how the structure might contribute to a naturally ventilated environment especially employing the thermal mass of the structure through its exposure. In general, typical floor to ceiling requirements might be in the order of 2.8 m to 3.0 m in addition to which the services and structural zone should be added to determine the floor to floor height.

Sensitive and often heavy equipment such as scanners and X-ray equipment will be required within a hospital environment. In instances where this may require suspension from the slab soffit or beneath a ceiling the engineer will need to ensure that a suitable fixing substrate can be provided and that the load and potential vibration are dealt with satisfactorily. Furthermore, there may be highly specialised equipment, for example, LINAC scanners, which may require specific design of the structure itself for the resistance of high levels of radiation used in treatment. The structure may need to incorporate additional protection in this regard and specialist advice appropriate for the particular equipment may be required.

Use of space	Response factor for continuous vibration
Operating theatre or precision laboratories	1
Wards or residential use	2–4 daytime
	1.4 night-time
General laboratories, offices	4
Other less sensitive areas, e.g. workshops	8

Table 8.1 Response factors for continuous vibration for use in hospital spaces. Data taken from NHS HTM 2045 (NHS, 1996)

Many areas in a hospital will be designated as being for 'clean' procedures. This in itself may not pose too many problems for the structure; however, it may impact on the use or location of downstand beams to avoid bulkheads or projections into these spaces which may otherwise create risk from infections or make cleaning or inspections more difficult.

Loading requirements for a hospital will in many instances be defined by and dependent upon on the user's requirements. Nevertheless, those loads (or actions) specified in guidance such as *Eurocode 1: Actions on Structures* (BSI, 2002) should be taken as the minimum for which design should be considered for the various medical, clinical or administrative uses. Additionally and invariably, specific and often heavy or highly sensitive equipment, both fixed or mobile (on transitory support) and either static or dynamic, may need to be considered; these being supported on either floor or ceiling.

8.3 Offices

The requirements for office space can vary considerably. Such use can include anything from 'a room with a desk' to high-rise commercial towers or large banking floors for blue chip international corporate businesses. The discussion presented here focuses on the larger provision of office floorplates within a commercial development.

One of the most critical considerations in establishing the plan form of an office is the efficiency of its configuration. This will be based on a number of considerations which can be summarised as follows:

- Efficiency of the floorplate in delivering the greatest lettable (usable) space in comparison to the area taken up by cores. This is referred to as the net to gross ratio and for economy and efficiency should be in the range 80–85% for low to medium rise developments of 5–20 storeys, reducing to a minimum of 70% for high-rise buildings in excess of 40 floors (see **Figure 8.3**).
- Depth of floorplate from windows in relation to floor to ceiling height. This forms the basis of how much natural light will extend into the floorplate and is defined as being economically optimised when it is at 2.0 to 2.5 times the floor to ceiling height or approximately 5.0 to 7.5 m into the floor from the edge. Increasing the floor height beyond this leads to an uneconomic cost of heightening the building through increased storey height and facade cost.
- Maintaining regularised comfort conditions in the space as a whole where perimeter space may suffer from solar heat gain (and therefore require introduction of shading or cooling) and areas beyond the perimeter zone needing to be maintained using artificial light and ventilation, all having an effect on energy consumption.

Therefore the floorplate design for an office will be driven by optimisation of its depth:floor height ratio so that natural light is able to penetrate deeper into the floor space. In Europe a narrow plate of 13.5 m is often preferred but this is considered to be less flexible since it limits options for usage layout when considering placement of corridors or aisles. In the UK (**Figure 8.3**), plate depths of 15–18 m are seen as ideal with ceiling heights of at least 2.8 m whereas in the US 18 to 21 m deep floorplates are common with 3.0 m high ceilings.

One of the essential components in determining an office layout is consideration of a 'planning grid' which is a combination of structure, fabric, services, ceilings and finishes. The idea is that the overall space provided could be subdivided into smaller partitioned compartment offices and be coordinated with other components. In the UK, a planning grid of 1.5 m is viewed as the ideal: 1.5 m units are used in the design for the possible location of partitions to divide up the space so that

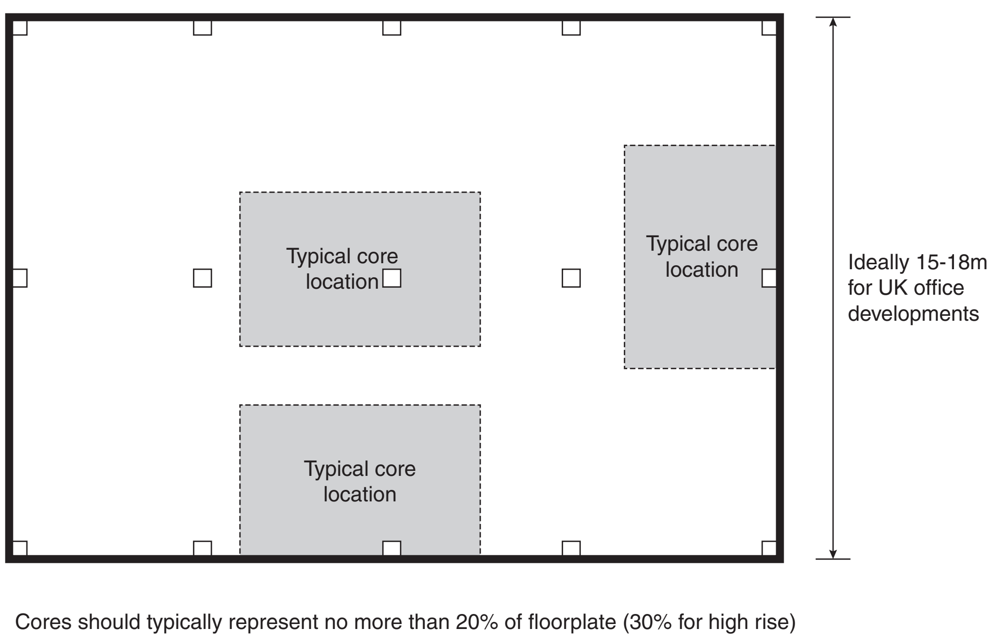

Figure 8.3 Ideal UK office floorplate arrangement

ceiling tiles or panels, glazing mullions, columns, lighting and services are delivered in an optimal way.

The structural grid should therefore be an integer multiple of the planning grid to allow unitary flexibility. The structural grid should be as large as economically possible, typically in the order of 7.5 to 9.0 m. Greater spans are likely to be only economically justifiable if occupancy type dictates the needs and the cost premium is justified by higher rental value. Nevertheless, column grids should allow for flexibility of internal layouts as allowance for change of use and layout leads to the best long-term sustainable solution.

A very good guide in the UK market, and the one commonly recognised as the standard guidance by which offices should be designed is the British Council for Offices' *Guide to Specification* (BCO, 2009). In the UK, the BCO guide became a milestone in the design standards for offices and particularly in respect of acknowledgement of realistic floor design loads. Prior to this guide, unrealistically high imposed loads were generally specified of 4 kN/m² or possibly 5.0 kN/m² along with a further partition load of 1 kN/m², often referred to as the 'institutional standard'. Following the research and discussion that went into preparation of the BCO guide it is now accepted in the UK and generally in the European office market that imposed loading should be based on 2.5 kN/m² with an additional 1 kN/m² for demountable lightweight partitions.

To allow office users to have designated areas of filing on each floor there is a further recommendation that 5% of each floorplate has an allowance for loading of 7.5 kN/m². This is often provided local to cores where higher loads can be accommodated more economically. Provision of raised floors (typically 150–300 mm deep) and false ceilings (with services therein) is now the norm in offices and the general recommendation is for an allowance of 0.85 kN/m² to be made (for the floor and ceiling combined) unless heavy data cabling or high services demand dictate a higher specific provision.

Building services will be a very important consideration in the design of offices both in the floor zones and ceilings. The approach taken to arrangement and type of services, particularly the heating and ventilation systems, will often drive the most suitable structural solution since the depth requirements for services involving ceiling ductwork in addition to the structural zone will determine the overall floor to floor height and therefore the overall building height and resulting elevational cost.

At the outset, therefore, it is vital that structural solutions are considered jointly with building services solutions. In concrete frames downstand beam arrangements are frequently discounted since they would hinder clear paths for services and require the latter to run underneath thereby substantially increasing the overall ceiling depth and floor height. Wide shallow downstands are a possible economic alternative. Concrete beams can be designed to permit the passage of building services but this will involve special detailing and fixing of reinforcement and formwork all of which will increase cost. A concrete frame and particularly the floor slabs may also assist with more sustainable and economic services and provide a cooling solution through the contribution of its thermal mass.

Steel beams with castellations or web openings may well prove an economic solution to allow services to pass through the downstand thus reducing the ceiling zone particularly for long spans where the equivalent concrete beam would not be as flexible in accommodating this. Similarly for ease of services routing, often after a structure has been completed, flat concrete slab or steel 'Slimfloor' or other proprietary shallow steel beam solutions that avoid downstands, whilst more expensive as an element cost, provide shallow overall floor systems in conjunction leaving an unhindered services zone which may in overall terms be more economic.

8.4 Retail

Retail developments take many forms from stand-alone high street shops, a parade of shop units, 'out-of-town' retail parks to shopping malls. There are two key drivers when considering the appropriate structural solution for all types of shop whether they are single units or large department stores, namely, flexibility of layout and the minimum or least obtrusive column spacing such that layout and maximum sales areas are not hindered.

The sizing of shops, where multiple units are to be provided, is based upon optimum trading frontage to depth of sales floor ratios. Units that are narrow and deep or wide and shallow do not provide for the space which is proven to be conducive to retail sales. For units with two trading floors from a mall or high street frontage the ideal frontage:depth ratio is proven to be between 1:3 and 1:4. For single storey units, the ideal ratio should not exceed 1:5.

The basis of the frontage width will therefore drive the column spacing. However, columns within the middle or even within the shop frontage are not desirable since they will hinder shopfront or access arrangements and of course attracting shoppers into the retail space is of vital importance if trading is to be successful. Columns close to the frontage may therefore need to be set back creating an edge cantilever to any slabs or roofs being supported. Likewise in the front zone often referred to in the UK as 'Zone A' (typically the first space of 6.1 m deep (20 feet) into the unit) with the highest floor area rental rate, columns will need to be strategically located so as not to become an encumbrance on trading layout. (Zone B is the next 6.1 m and Zone C the next 6.1 m with any further space being referred to as 'the remainder zone, space or area'; by comparison the rental rate for Zone C will be one quarter of that achieved for Zone A.)

In the UK and Europe, it is common to make reference to unit types based on size (trading floor area). Large shop units (LSUs) will typically be large department or variety stores. Whilst there is no set area for an LSU it will generally refer to a unit above 3500 m² to in excess of 12 000 m² for a full department store operator. A medium size unit (MSU) will typically be at least twice the area of a single shop unit and typically between 2000 m² and 3500 m² and will often extend over two floors and multiple structural bays, therefore having internal columns. A shop unit (SU) or more commonly 'unit shop' is typical of a single trading unit akin to a traditional high street shop with a single shop window frontage. Units of a smaller size only accessible from a small frontage and up to 10 m² in area will often be referred to as kiosks or 'lock-up' units.

Depending upon the type and size of the unit the storey height requirements will vary. Large stores needing to give the impression of volume as well as requiring a greater depth of ceiling for services (such as air ducts) will generally need to be provided with a storey height of 5.1–5.5 m and possibly more if large spans are required for the structure. MSUs and SUs typically require a storey height of 4.5 m or minimum of 4.0 m clear to structure. These are the heights which retailers require to merchandise the space and to maximise sales.

The spacing of the column grid therefore needs to take account of the variety of factors described above. By virtue of their large volume space LSUs will invariably require a large column grid. A common department store grid will be 12 m × 10.2 m, although some LSU retailers may require up to 14 m. Others may be happier to work with a 9 m × 9 m grid but rarely will it be less than this in an LSU. There may also be different uses above or below (for example, car parking) which may drive an optimum grid spacing.

The starting point for the ideal width of an SU is based upon a notional 7.0 m (23 feet) width, which will typically allow for an economical structure as well. The columns in a single SU will then occur in the division wall between units (some retailers may also require these to be flush in the wall without projecting into their unit). An MSU typically being a combination of multiple shop units will be based on a similar grid with columns occurring within the trading floor. In these units, the optimum frontage:depth ratio may also dictate the column grid.

In mall or shopping centre design, developers or owners require the shop units to have maximum flexibility of size so that shops of many widths and sizes can be created without too much hindrance from the structure. This is generally because at the time of design, tenants are unlikely to have been signed up to take space and even if they have, they may not define any specific requirements to be incorporated into the structure (for example, lift pits, staircases or escalators) until the structure itself has already been built.

It is therefore important that such requirements are kept in mind and probable or notional positions for openings allowed for at certain locations in the design. Ongoing flexibility of layout in future years is also important since turnover of retailers is common. As they grow or shrink or change sales practices (for example, through online (internet) trading and the need for goods to be picked up rather than displayed – known as 'click-n-collect') retailers often require different sized spaces.

Floor loading requirements in retail are commonly accommodated by an overall allowance of 5 kN/m² which will meet the majority of high street retailers' requirements for both sales and back-of-house (storage). Some may require a further allowance of 1 kN/m². LSUs may typically have higher requirements for storage or warehouse areas often at 7.5 kN/m². Plant requirements for regular shop units or even MSUs are not great since such units are not highly serviced typically being based on requirements simply comfort cooling provision (e.g., external condenser units). Plant loads of 4 kN/m² will often suffice in these instances. For LSU use, plant will be more extensive and may include heavier equipment such as chillers, air handling units and water tanks; therefore a typical plant loading of 7.5 kN/m² is the normal allowance.

In SUs (a typical unit shop) or MSUs there is often a requirement for the introduction of a mezzanine floor, either from the outset of construction or for later addition. This is a floor to be introduced between the ground floor and first floor soffit to create two floors of trading although the envelope is still regarded as being one storey. Historically the introduction of a mezzanine (typically by a tenant) neither attracted rent to the landlord nor affected business rates on the additional floor area. However, changes in legislation have affected this beneficial position in terms of rates, and landlords often now have rental leases based either on turnover or actual sales area. Nevertheless, structurally the load of such a floor, whether added from the first or later, needs to be allowed for along with some consideration of how such a structure might be physically (and safely) introduced later together with any effect it may have on the original structure (for example, foundations or loading or physical placement of load on supporting columns).

A further aspect of shopping centres is often their need to be developed for mixed use, the most common being car parking on roofs or in basements beneath. Leisure facilities, typically restaurants or cinemas, and residential uses are also commonly combined with a retail centre to bring about added value. The differing requirements for combined use become more difficult to accommodate when vertically stacked and inevitably differing column grids are required. Differing column grids invariably require transfer structure (typically beams) which will prove to be expensive. Consideration should therefore be given to how an arrangement might work whereby columns could be aligned throughout the height of the structure in part or throughout thus avoiding the need for transfer members or a transfer deck.

Provision of car parking beneath a retail centre is extremely common. These two uses can often prove to be the most compatible in terms of commonality of column spacing. A column grid of 7.5 m defined to be the ideal minimum in the width of a shop unit should simply require adjustment to 7.8 m or 8 m in order to align with a suitable car park column placement every third car park bay. Similarly in the opposing direction a column grid of 8 m (or 16 m) will suit both the retail and car park levels thus allowing continuation of columns through the heights of both spaces, avoiding the need for more expensive and deeper transfer structure.

Developments for food retail use, particularly supermarkets, require slightly different consideration to those for 'leisure' shopping. Supermarkets, defined as those having typical sales floor of greater than 2500 m² and anything up to 17 000 m² for the largest in the UK (and often more in continental Europe), will often be single-storey buildings although there is commonly the requirement for later mezzanine introduction to create two-level trading.

Storey heights for food superstores will often need to be as much as 5 to 6 m. Columns will need to be as infrequently and as unobtrusively placed as possible. In the food sales areas of single-storey superstores (with flat roofs over), containing the food rack storage 'gondolas', fridges and till banks, columns should be on at least a 20 metre grid or more. The structural solution will invariably be driven towards a steel truss arrangement supporting a flat metal deck type roof on purlins. Back of house areas such as the goods warehouse and staff facilities area would utilise more traditional construction of a framed arrangement with columns on a regular (typically 6 m × 6 m) grid.

Floor loads in a food superstore can often be specified by the food retailer as anything up to 15 kN/m² although 10 kNm² to 12.5 kN/m² or possibly less now is becoming the norm. In addition there may be racking or refrigerator point loads to consider. Both refrigerator area and checkout tills may also require recessed floor ducts within the structure which may have an effect on suspended floors particularly.

In food retail particularly, there is also a drive towards use of more visually sustainable structures involving the likes of timber (glulam) frames although this may not necessarily be the most economical 'first' solution on paper in comparison to the alternative steel framed solution. Some retailers are now automatically specifying that their single-storey food superstore should be designed in timber as this is now seen by the public as a 'reason to shop there'. This can readily be achieved using timber glulam members and it adds a very appealing aesthetic dimension to the interior of the stores. The typical grid used in these instances is 15 × 15 m or less and will adopt a combination of glulam beams and purlins along with timber columns as single posts or as 'tree' columns using strut branch legs.

8.5 Industrial buildings

There are many building types which could fall into the category of 'industrial' but this commentary will focus on those commonly referred to as industrial warehouses or 'sheds' typically of single-storey form. These buildings will typically be utilised for a light manufacturing process and assembly use or for material or goods storage requiring a large space volume of a utilitarian nature.

Such industrial buildings can vary from a few thousand square metres, the size of a tennis court, to tens of thousands of square metres, as vast as several football pitches. The average size of an industrial shed is in the order of 8000 m² (≈80 000 sq ft) but the upper limit may be in excess of 23 000 m² (≈250 000 sq ft). The largest in the UK is over 49 000 m² (≈530 000 sq ft) but such an extreme size is often a one-off to suit a specific need. In continental Europe and particularly the USA, large 'sheds' of this order are more commonplace.

If the land area is available, there would be no particular reason why the format of an industrial shed could not be adopted for whatever plan size of building may be required. The structure does not become any more complex (other than if ground conditions vary) as it would for a high-rise building where load at the foundations and wind forces increase with height. The same solution could simply expand horizontally with, of course, any due consideration of thermal movement allowance and provision of suitable bracing.

In the UK, the most commonly adopted industrial shed structure is of a steel portal frame form. **Figure 8.4** shows a typical portal frame arrangement. These will typically be of a steel UB section and have a sloping rafter with eaves and ridge (apex) haunches. Usually the roof pitch specified is between 5.0 to 7.0 degrees, 6.0 degrees being common, roughly 1:10. The frame spacing will be determined through an iterative consideration of a number of factors. These will include loading on roof, form of cladding and/or roof covering, internal use requirements, loading bay arrangements, span of roof and purlins/cladding rails and resulting depths, along with ground conditions and thereby foundations. Most portal frames will be economically spaced between 7.0 and 9.0 m.

An industrial unit is often specified (by the client) in terms of height requirements to eaves since the internal uses often require high volumes typically for use of high bay racking to make full use of height and the ability to go higher within the apex of the roof (into the ridge). A typical height of eaves, being the height from the ground slab to the underside of any portal frame haunch will often be 10 m in the UK although it can be higher. Such frames will be most economically designed using plastic design approaches and sizing of the rafter, column and haunch will be determined through a combination of bending and deflection both of the apex and the knee (eaves) and the ability for the building as a whole to accommodate this depending particularly on the type of cladding (for example, masonry or metal panelised cladding).

Spans of portals will achieve greatest economy if within the range of 18–30 m based upon a duo-pitch roof. Multi-bays creating large plan buildings will be formed of conjoined duo-span portals of a series of ridges and valleys to the roof. In a multi-bay arrangement it is common to omit every other column on a valley line through introduction of a valley beam spanning between two portal columns.

Another framing arrangement more often used in the USA and Europe, but less frequently in the UK, is a long-span steel truss system supported on posts. This is typically adopted with a 'flat' (1 degree pitch) roof and will result in the use of smaller and fewer columns within the internal space. However, the overall roof structure will be significantly deeper than the equivalent portal frame rafter with trusses typically being based on depths of span over 20 m for greatest efficiency and economy. Assuming a simple roofscape with high load requirements (e.g. plant or suspended equipment) columns could be placed at a typical spacing of 25 m. However, without portalisation

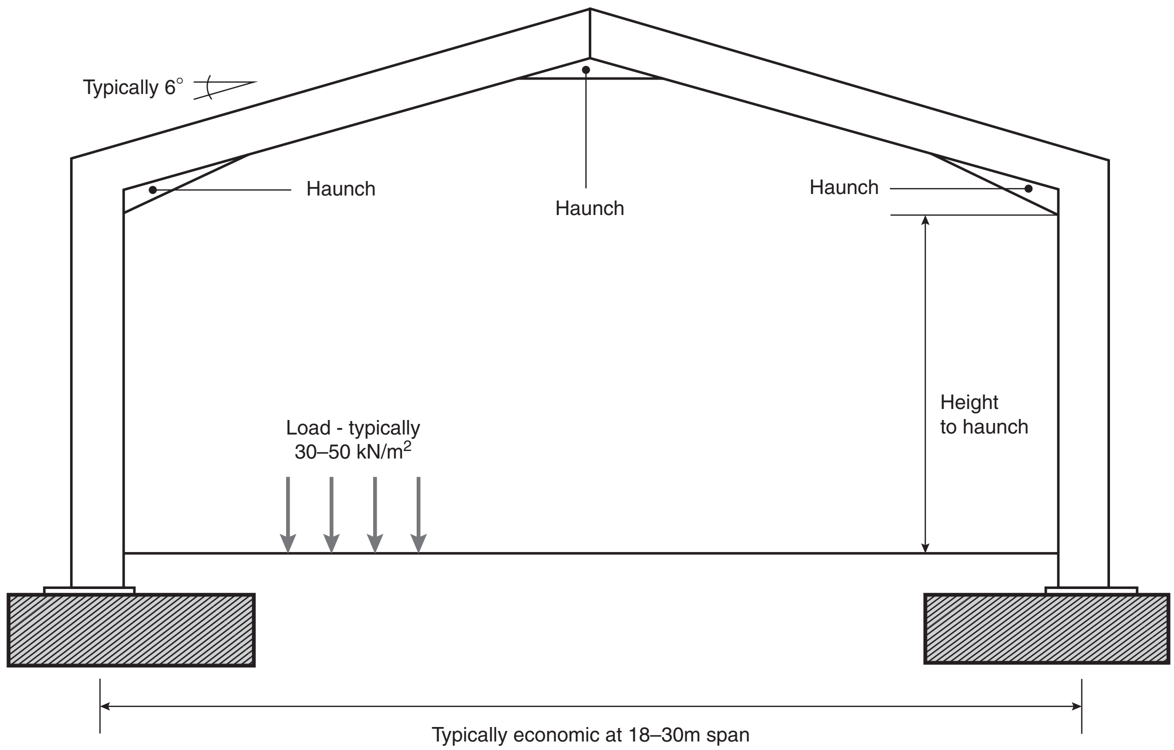

Figure 8.4 Typical portal frame arrangement

consideration must be given to strategic placement of bracing or some portalisation (moment connections) for the frame through the truss to column connections.

The nature of these buildings forming storage or manufacturing use requires high loading capacity for the ground slabs (not typically defined in any reference material). Whilst the end-user may have specific requirements, developments of this nature built speculatively or not without an end-user defined at the time are typically designed for loading of between 30 and 50 kN/m². In addition, the use of high bay racking will apply very high individual leg loads which must be sustained by the slab and often this will create a worst case for the slab design. Invariably such slabs will be need to be ground-bearing on proof-rolled (and tested) fill material to provide such capacity, as suspended slabs will most likely be incapable, within the bounds of economy, of being designed to accommodate the high floor loads and/or point loads. With a ground-bearing slab, consideration should also be given to placement or occurrence of leg loads relative to any construction joints.

It is also the norm in industrial buildings, and particularly so where high bay racking may adopt mechanised systems or fork-lift vehicles to access the goods stored on rack shelves, for slabs to be laid to very tight level tolerances especially in regard to surface flatness and regularity. Humps, troughs and slopes in a slab cannot be tolerated when used by equipment relying upon continuous verticality or plumbness which could otherwise topple over. It will be normal for these tolerance requirements to be clearly defined and specified by the engineer and require the highest level of construction quality to achieve. Contractors will often utilise the specialised laser-levelling slab laying plant that is available to achieve this high level of specification.

In the UK, the recognised reference guidance material in respect of industrial ground slabs is the Concrete Society's Technical Report No 34 *Concrete Industrial Ground Floors: A Guide to Design and Construction* (TR34) (Concrete Society, 2003). Engineering specifications for industrial floors will often make specific reference to this document and the criteria it defines in regard to slab tolerances to achieve the requirements for end-use.

8.6 Residential

In the context of residential buildings this section discusses typical design considerations for multi-storey blocks of multiple occupancies (e.g. apartments), rather than the traditional domestic house or terrace of houses. Specific design considerations which might additionally affect higher or 'high-rise' residential structures in excess of say 20 storeys are covered elsewhere in this manual.

Many of the structural considerations to be deployed in the design of residential blocks will ultimately depend on the intended market value and quality level of the individual development. For example, if the developers' target market is at the highly prestigious end, the particular brief may well override the normal economic bounds applied to structural design. Such considerations will therefore be highly specific and bespoke and cannot be discussed here; nevertheless specifications of clear-span large-volume spaces allowing the space to be configured as desired without interference from wall or columns, tend to be the minimum required. In these design briefs, the structure must not become an undue encumbrance to delivery of a client's requirements.

It is therefore more reasonable in this context to outline some of the normal considerations for the typical private market or housing association type residential apartment buildings which tend to be developed on the basis of 'volume build'. Speed of construction and repetition will therefore be an important consideration to the developer who will often be the contractor (or construction manager) as well, and therefore driving construction from the perspective of development value for maximum return. The quicker the development can be delivered to market the better and therefore speed and regularity of construction will become more important than the pure economic equation of the type of construction itself.

Design loading requirements will typically be at the lower end of any building loads. In addition to any particular finishes load, the imposed load requirements will generally be 1.5 kN/m² although in larger space areas and in apartment design a further 0.5 kN/m² can be required for lightweight movable or demountable screening or partitioning. Congregational areas such as terraces and shared lobbies may require greater load to be allowed for.

An important requirement within any residential use is the provision of natural daylight. Structure within external elevations should therefore be kept unobtrusive and downstand beams avoided or kept to a minimum. Upstands may sometimes be more acceptable where, for example, a perimeter beam may be required. Perimeter support structure (walls and columns) within or close to party walls may need to be checked for fire resistance since these are locations where horizontal fire spread must be considered.

Floor heights within apartments (excluding duplex double-height space) need to be proportional to reflect the depth of the spaces, and to optimise natural light into the rooms. Nevertheless the spaces should not become so cavernous as not to be cosy. The normal bounds are therefore deemed to be floor to ceiling heights of a minimum of 2350 mm to an ideal of 2700 mm to 2900 mm. Clearly decisions may be driven by maximising the number of floors in a given height for economic reasons or restricted by planning constraints but the more that internal height is compromised the more this may adversely reflect the market value of the residential units.

Structure that might involve internal downstands should be avoided in general and most certainly in large space areas such as living and dining rooms. Any downstands which are unavoidable should be strategically placed since they cause light shadowing and interference with light paths and are generally perceived as an inconvenience in residential structures. The most preferable location for any downstands is above defined division wall lines where a nominal bulkhead might be utilised if necessary.

The form of slab construction needs to be given very careful consideration particularly in the UK where compliance with Building Regulations Part E (The Building Regulations, 2010) for acoustic separation places stringent requirements on the floor not to allow transfer of noise between occupancies. These Regulations define tight acoustic (noise) resistance criteria that the floor build-up shall meet in the form of 'robust details' (those deemed as compliant if adopted). This usually requires the use of a thick dense concrete or the use of 'floating floors' specifically designed for acoustic separation. Alternatively, those construction details deemed not to align with robust details would require *in situ* testing but this brings about potential risk very late in a construction programme which is best avoided.

Proprietary forms of residential construction may also be considered for developments up to four or five storeys. These may take the form of both metal (cold-rolled) or timber framing and are lightweight compared to traditional house construction materials of masonry or concrete framing. Such systems become suitable for mixed-use developments where the residential element may be placed above other uses with large span requirements. These system-build forms of construction also offer greater sustainability credentials using materials with less embedded energy, may be fabricated off-site and can be quicker to construct.

8.7 Schools

Educational buildings may have many uses, from primary, middle or secondary schools to colleges and universities. The discussion in this section focuses mainly on multi-storey buildings developed for secondary school use although many aspects of the design considerations will apply to all.

Teaching methods and curricular content constantly change from year to year. New technologies have had a considerable effect on the space and the environment needs for educational buildings and advancement in this area continues apace. The structure designed for these buildings therefore needs to ensure that it would not unnecessarily hinder the ability for the building to adapt and create flexible space. The structure should therefore ideally be framed rather than rely upon any internal load-bearing elements other than those that would remain unchanged, for example, core walls.

Typically teaching spaces will not be heavily serviced within floors or ceilings. Indeed it may be common for there to be no finishes requirements for floors or ceilings (other than decorative); power to the spaces is typically provided within wall trunking, heating being in perimeter lines, lighting from ceiling fixed or hung 'rafts', and IT provision via wireless means or wall trunking. Floors can typically therefore be simply power floated to receive a vinyl-type finish.

Additionally, the internal environment and particularly the teaching spaces should benefit from maximisation of daylight (with appropriate limitation of glare). This would lead to the structure avoiding use of downstands particularly at the perimeter where if an edge beam is needed an upstand arrangement may be more suitable.

Naturally ventilated spaces in schools are the ideal and the structure can play its part in the provision of this. Without the need for ceilings the structure itself can be exposed to use the thermal mass of the slab for night-time heat purge cooling from the warmth built up in the concrete during daylight hours. Air flow through the classroom or teaching spaces can be assisted by passive means adopting air extract shafts (chimneys) at the rear of the rooms ventilating directly to roofs drawing air through the rooms from openable windows. The viability of natural ventilation solutions will require consideration of the external environment and the proximity of noise sources by a building services engineer, but the structural engineer should nevertheless actively contribute to the means by which the structure can contribute to the most sustainable solution to create a comfortable and economic environment.

Design loading will typically need to be that generally specified by local or defined educational authority standards together with reference to *Eurocode 1: Actions on Structures* (BSI, 2002). Any areas which might be the subject of dynamic loads, for example, gymnasiums, dance or drama rooms, should be considered in terms of any effect they may have on adjacent uses through connectivity of any structure.

All structural materials may be suitable for consideration in school and educational building design and the choice will depend on the specific requirements of the space and normal engineering design considerations in determining suitable structure. Framing solutions involving either steelwork or concrete or both could prove economically viable and should be assessed on their specific merits for meeting a particular brief. Equally, timber may offer attractive solutions for large span single-storey spaces such as gyms or assembly halls. The materials specified, however, should generally be of a robust nature requiring minimal maintenance in use.

8.8 Leisure

Many types of building can be defined as leisure facilities. These include swimming pools and sports buildings, stadia, cinemas, theatres, hotels and libraries and many more. In the following, we focus on the more popular of these, namely swimming pools, cinemas (theatres being similar to these in concept) and hotels. Stadia will typically be for one-off requirements and may require more specialised consideration for which specific alternative reference should be sought.

8.8.1 Swimming pools

In the context of this chapter swimming pools are taken to be those which form part of a community leisure facility and not those at a domestic property level.

Leisure facilities including a sports and/or leisure pool will typically be based around the swimming arena containing a six-lane (typically 13 m wide) or eight-lane (typically 17 m wide) pool of 25 m length for competition sports and leisure use or 50 m length if required to be of national or international competition pool standard. In addition, supplementary learning and leisure pools, spectator facilities and seating may be required alongside the main pool provision. The pool areas and any spectator facilities will require the provision of clear span structure to achieve the volume needed for the use. The structure provided for this will typically be exposed and often expressed as a feature structure. Whilst the structure will be led by the defined architectural requirements, the expression of aesthetic structure gives the engineer the rare opportunity to design and determine the specific form of structure that will deliver a pleasing appearance and effect.

Whilst the structural solution has many aspects which must be considered, not least the chlorine-based highly humid pool atmospheric environment, the options considered must be able to deliver the long span condition across the width of the pool and ancillary areas surrounding it, giving the user a feeling of lightness and volume to the space. The form of the structure is often required to be curved or sloping, simple pitched, saw-tooth or flat and should provide a minimum clear height of 3.5 m; rising from that where the roof is sloping. Should the pool incorporate platform diving boards then the height will need to increase accordingly and specialist advice should be sought in this instance.

In many cases, given the need for an aesthetically pleasing exposed structure, the use of deep glulam timber beams proves to be a highly suitable structural solution as well as being highly sustainable. Such members are also readily supplied with the curving profiles that may be desired architecturally. With suitably specified preservative protection, timber is ideal for use in a pool environment.

Equally, given the long span condition structural steelwork can provide a highly economical structural material and can be designed with aesthetics in mind utilising, for example, feature trusses or curved steel (solid or cellular) beam profiles. Use of steelwork will require careful consideration to long-term anti-corrosion paint system specification and a good reference for such specifications can be obtained from Tata Steel (formerly Corus) in the UK (Corus Construction & Industrial, 2004).

In the design of roofs over pools, to assist backstroke swimmers particularly, a linear visual point of reference along the roof, parallel to the lanes, should be provided which could be in the form of purlin alignment or roof light placement.

The pool 'tank' design and construction approach is of vital importance. The pool tank can either be constructed *in situ*, in forms such as reinforced concrete, tanked and rendered reinforced masonry, or in a proprietary prefabricated type such as those now manufactured as stainless steel panels bolted together on site and laid on a base slab. Whatever the form of construction it is vitally important that the design is based on water-retaining standards (e.g. BS 8007 for concrete design in the UK) and that the construction is carried out to a high specification level and quality control regime. Water-tightness is paramount from the dual perspectives of keeping water in but also keeping any external groundwater out so as not to cause any contamination. Testing of water-tightness before construction finalisation and particularly backfilling and application of finishes is essential.

The structural design of any retaining wall to the pool tank should also take account of a temporary free-cantilever condition since backfill is likely to be placed (thus surcharging the wall) before the pool surround slab is cast to provide ultimate propping restraint. The tank should also be considered in design terms for it to be emptied during a water purge which happens during a maintenance cycle. External ground and groundwater conditions may prove to be a worst case design consideration on the pool tank when it is void of water. The tank construction will also include significant ducting and trenching requirements for the flow of pool water from and to the filter systems and to backwash tanks which must be allowed for in the design approach and detailing.

The main point of reference for building design guidance in respect of swimming pools focused on the UK, but equally applicable for guidance in any country, should be Sport England's *Design Guidance Note: Swimming Pools* (Sport England, 2011). Every aspect of swimming pool building design is presented in this excellent guidance booklet.

8.8.2 Hotels

Hotel use of a building is akin to residential use in a transient commercial sense. The specification for a hotel will be very much determined by the budget level at which it is to operate in the market.

Hotels will therefore vary from the budget level targeted towards a cheap and pleasant overnight stay to the five-star boutique level offering the height of luxury and an accommodation 'experience' where the expense of the accommodation is not so important to the user but the level of luxury is.

A development specification for a budget hotel will be driven by the economy of the overall solution and its ability to be delivered rapidly. Given the lighter (imposed) loading requirements akin to residential loading levels typically at 2.0 kN/m^2 there is the ability to utilise almost any structural material. This type of building will typically be highly regular in plan form to deliver a 'standard' room fronting external daylight facades and repeated many times over.

To assist with rapidity of construction prefabricated systems are often adopted utilising cold formed metal framing panels and floors with 'c' and 'z' section members. These panelised systems can even be prefabricated to include surface finishes, insulation and 'first-fix' services elements such as electrical wiring and pipes for water supply. These panels are then 'bolted' together as a kit of parts on site. These framing systems rely upon the division walls between rooms and sometimes corridors to be main load-bearing vertical support elements. Such system buildings are typically economic for use in up to five-storey structures due to load-bearing limitations and the additional design criteria for consideration of disproportionate collapse. System-build type construction for hotels may also incorporate other prefabricated components such as bathrooms delivered in 'pod' form for simple placement into the frame.

Other forms of construction typically adopted for hotel buildings will include concrete which on a similar basis to that discussed in the foregoing residential section will require avoidance of perimeter downstand elements, which in this case might hinder placement of items such as bathroom pods into the floorplates.

The design of all forms of structure, whatever the level of specification, must ensure that due consideration is given to avoidance of noise transfer vertically and horizontally. Whilst in the UK there will be requirements in the Building Regulations which must be met with regard to design, specific hotel operators may have additional requirements set at more stringent levels.

For hotel buildings seeking to operate in the mid-range to luxury market, the specification requirements will very much depend upon the bespoke needs of the hotel operator (end-user). Each hotelier will have specific requirements that may often apply across international markets such that a guest knows that the standards they expect across the 'brand' are applied wherever they might stay in the world. The design will therefore be wholly led by delivering to a defined brief set by the operator.

8.8.3 Cinemas (and theatre auditoriums)

In the marketplace today it is common for cinemas to be developed as multi-screen operations referred to as multiplexes, sometimes providing for 16 or more individual auditoriums. These multiplexes also provide for a variety of auditorium sizes to accommodate differing audience attendance levels over the course of a film's showing period or its popularity. In addition to the auditorium the building will also contain public foyer assembly areas and film projection and access galleries as well as sales concession spaces. In design terms, the principles of multiple cinema auditoria requirements would apply singly to a theatre auditorium.

The first consideration for a cinema should be the design solution that will provide for large clear spans of either a single or multiple spaces. Auditoriums may vary in width between 12 and 18 m or more and up to 10 m high. Structure that will achieve such clear spans economically will typically involve structural steelwork rather than concrete and could include consideration of regular steel beams, trusses or cellular beams. The roof structure and finishes especially should also take account of the need to avoid rain drumming affects and noise transfer in the space beneath.

Depending upon the size of the auditorium, spectator seating will either be flat, sloping or terraced, over steps also known as bleachers. The sloping floors can take many forms depending upon the height of rise from the screen or stage line to the rear of the auditorium. A low rise slope is often formed of a void filler such as polystyrene with a concrete slab cast over, alternatively timber flooring can be utilised. Where the void becomes greater and where the space beneath the top of the rise would achieve usable headroom a suspended structure can be utilised. This could take the form of precast 'bleacher' steps sitting on and spanning between sloping steel beams beneath.

Of key consideration to the design of the structure as a whole is the limitation of acoustic vibration between elements of structure both within any one auditorium and that which might cross between adjacent auditoriums. It is of vital importance that all structure is assessed for provision of acoustic separation between auditoriums. One auditorium may experience high noise levels during the showing of a movie (and thereby reverberation and vibration) while there is silence in an adjacent one. There can be no cross-effect experienced through the structure.

Acoustic isolation through structural separation, independence of individual auditorium structure or specification of specific acoustic bearings may be required. The internal structure, for example the seating terrace structure, may need to be designed so it is isolated from the primary structure on which it sits to avoid transference of reverberation. Specialist advice from an acoustician should be sought by the client to suitably inform the engineering design.

8.9 Conclusion

This chapter has attempted to demonstrate to engineers who are developing designs to fulfil a client's brief for a particular building, the differing aspects they should take account of, duly consider and attempt to deliver to meet best design practice and provide an economic solution. While not every type of building has specifically been covered in this chapter, from the guidance given for the more typical building uses, an engineer will begin to understand that there is no 'one size fits all' solution.

Every building to be delivered for a client will, in its own way, be unique, although there will be key considerations dependent on its end-use which the engineer will need to know and the client will likewise be expecting.

This chapter has sought to inform the engineer on the key considerations and design solutions they would need to deliver for a variety of building uses they are likely to come across in their everyday design life. The fundament principles outlined for each building type can therefore be applied when a new design is being developed for any building a client is likely to require.

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CONTENTS

Chapter 9

How buildings fail

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This chapter introduces the concept of building failure, covering the methods of failure commonly seen such as progressive collapse and failure due to ongoing serviceability issues leading to deterioration of all, or part, of the structure. It discusses the requirements of suitable building foundations and the cause and effect of external environmental issues on the substructure and the requirement for adequate pre-construction investigations. The analysis and design of buildings will be discussed and this will review the requirements of material suitability, ongoing structural stability and the knowledge/experience of the designer and checking engineer. Material failure will also be covered, including deleterious materials along with long-term serviceability issues associated with common building materials and their degradation which can ultimately lead to building failure.

9.1 Introduction

Building failures/collapses may be attributed to poor design, faulty construction, foundation failure or other factors, which may be unknown at the time of design and the subsequent construction.

Depending upon location, the building designer is responsible for adequately assessing the parameters (or loadings) that the building may be subjected to, whether they are the working loads for the proposed usage, fire safety criteria or extreme environmental conditions such as earthquakes, hurricanes or extreme temperatures. Designing to support such parameters without excessive deflection or, more importantly, initiating a collapse mechanism is a skill that is gained over many years of training and experience.

Sudden catastrophic failure of buildings are thankfully uncommon within the UK and this is due to the experience of qualified engineers and the use of codes and building standards extensively developed over the last 50 years.

There are still however, buildings which fail in a non-catastrophic way, i.e. without complete or partial collapse. In these cases, minor faults, usually derived from serviceability issues, occur which if not dealt with early may lead to prolonged deterioration and eventual failure of a structural member or system of construction.

9.2 Types of failure

Full structural collapse is uncommon in the UK and usually occurs following an unusual event such as explosion, uncontrolled demolition or uncontrolled structural alterations during refurbishment.

An explosion within, or adjacent to, a building creates additional pressure loads on structural members which may, or may not, have been designed to withstand such forces. As the blast pressures are not sustained loads but high applied loads acting in both positive and negative directions on a member, these members can become overstressed in a number of directions causing the members to fail.

Uncontrolled demolition is the effect of the structure falling to ground in an unexpected or unmanaged manner. Either of these collapse situations would lead to the imposition of additional loads, i.e. the weight of collapsing material on other parts of the structure which then fail due to overloading and subsequently increase the rate of the demolition with little or no control on the extent or collapse area. This is known as progressive structural collapse.

9.2.1 Progressive collapse

The largest failure of this kind in the UK occurred in 1968 with the Ronan Point collapse where a gas explosion on the eighteenth floor of a 22-storey precast concrete building blew out the opposite corner walls which provided the sole support for the walls above (**Figure 9.1**). Consequently, the domino effect of the collapsing hollow core concrete floors onto the floors below caused the second phase of the progressive collapse

Figure 9.1 Progressive collapse, Ronan Point, 1968. All rights reserved

down to ground level. In this case the extent of the damage was disproportionate to the cause. Four people were killed and seventeen injured in this collapse of the partially occupied block; a figure that could have been significantly higher had the building not only just opened so the majority of flats above the affected floor were unoccupied, and had the explosion not occurred in the early morning when most people were asleep, with the collapse shearing off the living rooms from the remaining flat floor area.

The construction of Ronan Point was of a factory-built precast concrete system, designed by Larsen-Nielsen and developed in 1948, and was the second of nine high-rise structures constructed to aid rehousing in London following the Second World War. This system build relied upon the floor and wall panels being bolted together within slots with the joints then being packed with dry mortar to secure and seal the connections.

It was found during investigations following the collapse that the panel fixing details were inadequate and it was estimated that the exterior wall was moved at a gas pressure of 3 psi (21 kPa) (Griffiths, 1968). It was also found that the tower block was seriously underdesigned for the high wind loading which could also have caused progressive collapse of the structure.

The structural investigations also revealed a catalogue of construction errors/omissions which created gaps between floor and wall panels thereby providing fire or smoke routes between floors. It also highlighted the lack of mortar packing between joints, which enabled the building to rock on its connections during high winds.

Overall, the structure could have failed in a number of ways and it was only a matter of time before a full collapse failure occurred. The thorough investigations revealed that the collapse of Ronan Point was due to its lack of structural redundancy. The building had no fail-safe mechanisms and there were no alternative load paths for the upper floors should a supporting element give way below.

Following the Ronan Point collapse the UK Building Regulations and subsequently the concrete design code at that time (BS CP110–1972) were revised to ensure that structures with five or more storeys were designed against progressive collapse. This was followed a number of years later by the American Concrete Institute (ACI) which developed its concrete code ACI-318 to include design provision for the failure mechanism.

Progressive collapse is a particular risk where building alterations are being undertaken involving the removal of structural walls or laterally restraining floors. In these cases the structure is not adequately restrained whilst works are being undertaken thereby changing the load path and causing overstressing of the existing members.

A more common form of collapse is that of freestanding walls which may have been inadequately designed, such as having an incorrect width/height ratio or being subjected to excessive lateral loadings.

Further discussion on design against disproportionate collapse is given in Chapter 12: *Structural robustness*.

9.2.2 Lift slab construction

Another system of construction used in the past was the lift slab construction technique, which was used for a number of structures including high-rise blocks and multi-storey car parks (MSCP). This system comprises precast concrete columns erected in pockets with the concrete floor slabs cast at ground level. These slabs are then jacked up the columns and lowered into position, locking off at each floor level with the use of wedges packed between the column face and the welded angle shear collars cast into the slab.

In 1997, a 120 tonne section of Pipers Row MSCP in Wolverhampton collapsed, fortunately at 3 a.m. leaving no casualties. The cause of the collapse was found to be punching shear failure of the slab at one column head, leading to progressive collapse as similar failures occurred at another eight column positions supporting the top car park level (**Figure 9.2**).

The car park had a history of surveys and local concrete patch repairs. However, the subsequent detailed investigation following the collapse revealed that the degradation to the top surface of the slab had led to a loss of reinforcement bond which led to a critical reduction in slab shear strength. The investigation also highlighted discrepancies in code-based shear strength calculations and assessment of existing structures especially after long-term water ingress and subsequent material degradation.

9.3 Foundation failure

During the construction of buildings, all soils will compress slightly as the soil density increases until the load-bearing capacity reaches equilibrium with the applied load. This compression or initial settlement will not necessarily be measurable during the construction phase and will usually be complete after the first few years following construction. This depends upon the type and form of foundation construction used. See Chapter 13: *Soil–structure interaction* for more details on load-bearing capacity and settlement.

Foundations are classed as shallow or deep depending upon their type and depth of construction. Shallow foundations are typically strip (spread) footings, pads and rafts and are used where the ground investigation has proven that the strata at shallow depths is competent and has sufficient bearing capacity to accept the building loads without excessive long-term settlements. It is unlikely that shallow foundations would be used where the ground consists of soft clays, silts, and peat or fill material without some form of ground improvement technique being undertaken.

Deep foundations are typically formed utilising piles, piers or caissons to transmit loads to more competent strata at depth. These foundation types use driven or bored methods to construct deep slender elements to a sufficient depth where the loads can be transmitted either directly to rock strata via end-

Figure 9.2 Punching shear failure at Pipers Row car park, Wolverhampton © Dr J Wood

bearing or using a combination of end-bearing and skin friction where vertical loads are resisted by the ground/pile surface friction along all or part of its length.

Foundation failures can usually be identified by the deformation or other signs of distress noted in the superstructure. Settlement of foundations (excessive ground movement) is highlighted by out of plumb walls, sloping floors and, where differential settlement occurs, stepped cracking in both internal and external masonry.

However, the type of foundation failure itself depends upon the type of foundation used in construction. In the majority of cases within the UK, failure may be caused by the lack of (or reduction in) ground-bearing capacity once the structure is complete and fully loaded. In some cases the foundation structure itself may fail, reducing the bearing area required to safely transmit the forces, such as shear failure of wide strip footings or achieving a false set when driving piles to reach a required bearing depth.

The pile member may fail due to shear failure of the surrounding soil and/or excessive settlement of the pile. This can also be confused with a lack of bearing on bedrock, as was the case with a project in Blackpool, Lancashire. Here, a small development of residential properties constructed on a sloping site was eventually part-demolished following failure of the piled foundations. In this case, the steel-cased piles were driven to found on rock; however, subsequent investigations discovered that a number of piles were bearing on a sloping outcrop causing a rotational failure of the loaded piles.

Failure of shallow foundations is typically found at positions of high building loads, areas of soft substrata or indeed both. These highly loaded areas include chimney stacks/breast in older properties and extended or refurbished buildings which have had increased structural loads such as additional floors or changes in load paths. On internal inspection of these properties it is common to observe that the floors, door frames and window sills slope in the direction of the settlement with associated structural cracking both internally and externally.

Raft foundations, which are designed to balance out the structural loads over the plan area of the building, are particularly susceptible to increased loads or soft spots on the perimeter of the building. In these cases the whole building may rotate on the raft without the associated external deformations; however, these movements may damage services into and out of the building.

Foundation settlements can occur following increases in groundwater beneath foundations. It is normal during the design phase when assessing the allowable bearing pressure (ABP) of granular soils to reduce the allowable pressure under the foundation by half when near to the water line. If water was not a consideration in the foundation design, either by the designer's choice, error or lack of investigation when assessing ABP, the addition of water to the soil strata beneath the foundation can reduce the effective density of the soil leading to a reduction in the factor of safety and ultimately double the settlement.

On the same basis, leaking drainage systems, water supplies, etc., may remove fine soil particles over a long period without the knowledge of the building owner until signs of settlement or heave are observed, usually in the form of cracking in the building fabric.

The leaking of drains adjacent to buildings tends to occur at joint positions rather than as a result of the failure of the pipes themselves and leaks are commonly found at pipe junctions such as gullies or soil pipes. Tree roots can also break through house drainage, thereby causing the escape of water and subsequent loss of fines and soil strength.

9.3.1 Mining

Building in mining areas is of particular concern and extensive desktop studies are required when undertaking projects in these areas. Records are available from various organisations including the Coal Authority, Brine Boards and the British Geological Society which may indicate the age, depth and extent of mine workings in the area, the shaft positions and any remediation measures undertaken in the past. However, it is essential that adequate site investigation including rock drilling is undertaken as part of any proposed development.

The type of mine workings may be dependent upon the mineral being extracted and these vary around the country. Of particular concern to buildings is the use of the pillar and stall method in shallow workings as used from the mid-fifteenth century to the nineteenth century, where the term 'shallow' refers to coal seam working within ten metres of the ground surface. Here, the mineral (usually coal) was extracted leaving pillars of coal to support the roof of the working which over time can deteriorate and cause a collapse of the roof and ground above.

9.3.2 Tree action

Buildings with trees located in close proximity are at risk of damage due to the action of tree roots or more commonly, water absorption. The shrinkage and swelling of clay soils is the most common cause of damage to older buildings within the UK costing the insurance industry £350m per year (Driscol and Skinner, 2007).

Trees extract varying amounts of water through their roots during the main growth in spring and summer. In clays, this results in shrinkage in both the horizontal and vertical directions, which if adjacent to a building, will cause damage to the foundations and building fabric. The extent of water absorption is wholly dependent upon the tree type/size and the clay soil properties as clays with a liquid limit greater than 50% are liable to have a high shrinkage capability.

Research indicates that evaporation may reach 0.5 m in depth with grass and short vegetation extending this drying range to 1.5 m; however, some of the larger trees can extract moisture to depths of 5 m or more (IStructE, 2000).

Following times of high or long-term water absorption and evaporation during summer, a wet winter can cause a swelling in the clay which can cause as much structural damage as the shrinking clay. This clay heave can apply vertical pressure to ground-bearing slabs and supported walls as well as a lateral force to buried masonry walls and concrete trench foundations.

It should be noted that where trees have been removed prior to construction, or even heavily pruned, the effects of soil heave are still possible as the desiccated clay rehydrates as the tree roots die away. The effects of this rehydration are well known and there are numerous design guides from sources such as the National House Building Council (NHBC) and the Building Research Establishment (BRE) where extensive investigations have been undertaken and effective details produced to combat the effects of these soil actions.

Foundations on unconsolidated made ground (fill) can also lead to ground movement over time and this can be exacerbated by vibrations from traffic or nearby construction works such as piling. This 'vibration induced' consolidation occurs in granular soils; therefore it is unlikely to cause damage in clay soils unless the clay has a high sand content.

9.3.3 Site investigation

As discussed previously, it is important when commencing a project that a desktop study is undertaken prior to setting the parameters of any geotechnical site investigation work. This will establish the history of the area and give adequate clues as to any potential problems such as historic ponds, clay pits, mining or industrial works on the proposed site. The need for adequate investigatory works is necessary for all projects and is part of the engineer's duty of care towards the client and also extends to the public. The client should be aware of all the material facts that could increase future costs during design, construction and beyond.

On smaller projects this element of advance investigative work may be costly and, regardless of the aforementioned duty of care, the client is often unhappy to pay for the engineer's time and that of a site investigation team. However, it is imperative that the designer should gain as much relevant site information as the budget allows in order that both the designer and client are protected against future costs or claims. Many local authorities hold information such as survey maps, aerial photographs and previous site investigations around the area and many of these sources can be viewed at little or low cost.

The basis of the investigations should be to the relevant Code of Practice for Site Investigations (BS 5930:1999) or meet the requirements of the relevant foundation design code.

This has now been superseded by the National Annexe to Eurocode 7: Geotechnical Design, Ground Investigation & Testing (BS EN1997-2:2007, published in 2009. See Chapter 13: *Soil–structure interaction* for a more detailed discussion of site investigation and geotechnical design.

9.3.4 Poor interpretation of results

In cases where a site investigation and study has been undertaken, the results are to be reviewed by a competent engineer with sufficient relevant experience dealing with soils and geotechnical engineering. This interpretation should provide at least a summary of the ground conditions on the site, the material strengths found during the site investigation works, such as standard penetration test (SPT) counts and the subsequent laboratory results providing such information as clay shear strengths, angle of internal friction and moisture content.

All this information would be utilised by an experienced engineer to choose the correct foundation type and design the structure accordingly, limiting settlements due to the building structure. Unfortunately, this work is sometimes passed to an inexperienced engineer and it is here that deficiencies in the design are found to occur. When the engineer does not have the relevant experience the factual investigation should be supplemented by an interpretive report by a specialist engineer, providing the necessary information along with foundation options and expected settlements.

Care should also be taken when extending properties ensuring that the new foundations do not rest or impose additional loads on to the existing foundations. New foundations should be adjacent to the existing structure and founded at a lower formation level. As it is common for new extensions to suffer from differential settlement, care should be taken when excavating the foundations, ensuring that firm strata is achieved and also that the superstructure is detailed in such a way that any minor settlement does not adversely affect the building fabric.

9.3.5 Choice of appropriate foundations

By utilising the site investigation reports including trial pits, the appropriate foundation type can be chosen. Soils with high settlement rates such as soft clays, silts and peat require a deep foundation such as piles to transit the loads to more capable strata at depth; whereas firm strata at shallow depths can readily accept normal foundation loads using strip, pad or raft foundations as long as this strata is consistent for depth and the pressure bulb does not lie in a softer lower strata.

In some cases where the proposed building loads are slightly higher than the allowable bearing capacity it may be possible to utilise ground improvement techniques such as vibro stone columns. This is a cheaper alternative to piling, especially over an area or number of plots, and would provide an improved bearing capacity for a shallow foundation type. Care should be taken with this type of ground improvement giving due regard to soft strata bands such as peat that provide little lateral restraint to the stone column, leading to bulging within the soft band layer.

9.3.6 Seismic/dynamic foundation failure

In order to assess the seismic response of a structure when it is located within a known area of seismic activity the knowledge of the soil strata composition is critical as the soil layers provide differing damping conditions which affect the amplitude of the ground motion applied to the structure.

Understanding the structural form and good knowledge of the site is essential when designing adequate foundations to transmit the building forces to suitable bearing strata. However, the damping conditions and amplitude also have a bearing on the design of the foundations with the aforementioned damping effects acting as springs against the buried structure or pile model.

The material characteristics of the strata layers are to be suitably assessed as part of the site investigation and laboratory testing and considered along with the choice of foundation system. For instance, saturated non-cohesive soils (sands and silts) under cyclic loading, as experienced in earthquakes, can become liquefied, a process called liquefaction (**Figure 9.3**).

Within saturated soils the pore water pressure is generally low; however, during the earthquake shake, the pore water pressure increases enabling the soil particles to move. As this occurs the soil strength decreases, creating a bearing capacity failure of the structure. This also affects pressures on buried

Figure 9.3 Liquefaction failure – Niigata earthquake, Japan 1964. Photograph in the public domain

structures such as retaining walls when, although designed to accommodate lateral loads from soil and water over the retained height, the increase in pore water pressure can cause rotational and sliding failure. As saturated soils are more common close to oceans, lakes and rivers, failures of harbour quays and bridges due to liquefaction are common during seismic events.

9.4 Modes of failure

In order to prevent disproportionate collapse, as previously discussed, it is essential that the building structure is adequately connected thereby providing the required restraint and robustness in the event of loss of support. These requirements are now well established in the Building Regulations (Section 5 A3) and advice is also given in the relevant design standards and construction guides.

Within the Building Regulations, key elements are highlighted, namely those structural elements which, if removed, would result in the instability of the building or damage in excess of the limits provided. These key elements are required to withstand accidental design loading, both vertically and horizontally, simultaneously with one third of the normal characteristic design loading.

The use of precast concrete panel systems has become more prevalent, especially following the Egan Report *Rethinking Construction* (Egan, 1998), and the drive to reduce waste and streamline the construction process has led to an increase in modular systems and off-site manufacturing. The key now, however, is the requirement for effective tying of structural members to prevent disproportionate collapse.

With traditional masonry wall construction, the floors and roofs provide some restraint to the external walls by either direct support/embedment into the wall or by the use of galvanised straps to provide the lateral restraint. Bulging may be the consequence of differential expansion/contraction between the inside and outside of the wall, sometimes in conjunction with wind pressures acting on unrestrained panels causing the outer leaf (of a cavity) to displace laterally, usually at floor levels. Here, the engineer should ensure there are adequate straps and cavity ties wall at these levels.

When designed to the relevant design standards, structural members should be able to resist the stresses due to applied loadings, whether static or dynamic (wind/seismic). These loadings may be applied to the structure in any direction and the effects of bending, shear or torsional failure of the structural member should be assessed to ensure the material is not subject to overstressing.

9.5 Material failure

Structural failures are commonly linked to the materials used in construction either due to long-term deterioration of the material or by overstressing the material in its permanent state.

Lack of knowledge of the material by design engineers is a common fault, especially where new materials or manufacturing methods are being used for the first time with the associated unknowns. In these cases, issues such as the true allowable material stresses or the inclusion of faults during the production process come to light. This highlights the reliance of the design engineer on adequate industry testing and academic research or analysis, and on bringing this to the engineering practitioners. As Hossain (2009) points out, ‘well designed structures, coupled with the hard effort of the experts and correct materials can ensure the structure a complete success’.

As has been discussed previously, the long-term deterioration of *in situ* materials is a common fault in buildings and this ties in with the requirement for proper management of the structure. Ongoing regular inspections of the structure to ascertain deterioration or degradation of the material is crucial, along with adequate material testing where required to ensure that the change of material properties does not affect the requirements of the structural system, as was shown in the Pipers Row MSCP collapse.

It is not just with new structures that we should be aware of possible long-term material faults. In the UK, we know of deleterious materials used in building construction in the past, which we would no longer regard as acceptable today. Examples of these materials are:

- High alumina cement or concrete
- Woodwool slabs as permanent formwork or in structural elements
- Concrete or mortar additives containing calcium chloride
- Aggregates for use in concrete (plain or reinforced) which do not comply with the relevant British Standard Specification
- Calcium silicate bricks or tiles
- Asbestos
- Lead

Of this list, some are materials found to be extremely dangerous to human health. For example, asbestos (present in insulation boards/cladding/tiles), if damaged, can release fibres into the air causing damage to the lining of the lungs and can lead to asbestosis or cancerous malignant mesothelioma, for which there is no cure. It is not purely a historical legacy of the construction industry as asbestos could be present in any building built or refurbished before the year 2000. However, in good condition, undisturbed and properly managed, asbestos does not pose a significant health risk (HSE, 2010). It is, therefore, essential that engineers, technicians and other surveying staff are aware of the risk of asbestos and should always ensure that if a building does not have an existing asbestos register that a suitable testing programme is undertaken before any investigation or construction is undertaken.

Also in the list, however, are deleterious materials which can cause or lead to structural failure in buildings. Specifically, high alumina cement/concrete (HAC) has been the cause of a number of building failures in the past due to changes in its characteristic material properties.

9.5.1 Concrete

HAC was made using calcium aluminate rather than the normal calcium silicate used in Portland cement (PC) and was popular due to its rapid development of strength, partly due to the increased proportions of aluminate instead of PC. It was used extensively in the manufacture of precast pre-stressed concrete beams for a 20-year period up to the mid-1970s, before the collapse of precast roof beams at three educational establishments led to it being banned from use in structural concrete.

Investigations found that the chemical properties of the calcium aluminate cement 'converted' over time which led to a loss of strength; ultimately the faster the rate of conversion, the greater the loss of strength (BRE, 1981). However, ongoing research over the past 20 years has determined that the primary cause of the collapses was due to poor workmanship exacerbated by the loss of strength due to the chemical conversion which, in some cases, also led to increased chemical attack.

Today, it is estimated that there are up to 50 000 buildings in the UK with similar beams constructed using HAC concrete and they continue to remain serviceable with no undue effects on the structure. Many are to be found in public buildings such as schools and other council building stock as well as older industrial buildings. The engineer should be aware that buildings of this age may still contain this material and adequate chemical and laboratory testing should be carried out to confirm its current state and ongoing suitability. The common use of a percussion hammer to test the strength of concrete is not recommended for suspected HAC concrete elements as the outer layer of the concrete may remain hard whilst the inner core material of the element has a reduced strength. Therefore hammer tests would be misleading and suggest a safe beam of suitable strength.

Whilst much has been said about HAC and the conversion of its properties, studies have been ongoing by the BRE regarding the long-term durability of HAC concrete elements in existing structures. The key durability issues (Dunster *et al.*, 2000) are carbonation of the concrete cover to reinforcement and chemical attack of the cement paste matrix by ingressing sulphates or alkalis. Many of the findings so far indicate that the deterioration of HAC concrete is similar to that of Portland cement concrete. The location of the HAC units within a building is of critical importance and it must be determined whether beams are dry or embedded in external cavity walls which may have high levels of moisture leading to increased corrosion of reinforcement, generally reacting in a similar way to Portland cement concrete. Should the HAC beams be located within a permanently leaking roof structure with alkali or sulphate ions from screeds or gypsum plaster, then this can lead to enhanced deterioration due to chemical reaction and significant loss of strength greater than would be expected with a Portland cement concrete.

Concrete, when designed and constructed properly, can have a significant life beyond that of the intended design life of the structure. However, serviceability issues and ongoing lack of maintenance may lead to early deterioration requiring repairs to defective concrete. The initial defects within a concrete structure, other than the design load effects, are generally due to the volumetric effects of the concrete such as: plastic settlement or shrinkage, early thermal contraction, long-term drying shrinkage, shrinkable aggregates, surface crazing and thermal movement.

Any of these effects may initiate cracking in the concrete surface providing channels for both water and air to reach the reinforcement, which over time will lead to corrosion of the steel and eventual spalling of the concrete cover.

There are many other causes of defects in reinforced concrete structures that should be taken into account when assessing the strength and long-term serviceability of the structure. These are not limited to cracking of the concrete but may over time affect the structural integrity of the concrete members and the overall building. These include: chloride contamination, alkali–silica reaction, sulphate/acid attack, efflorescence, lime bloom and atmospheric pollution. Carbonation of the concrete along with chloride ingress is one of the main reasons for concrete deterioration and the failure of reinforcement in concrete members.

Care should be taken when designing and detailing reinforced concrete buildings to ensure long-term durability to prevent, wherever possible, these causes of deterioration from occurring. The designer should be aware of the effects of these defects and the measures within the design standards to ensure they are covered. The designer should also take into account the effects of shrinkage and creep deformations which, if restrained, will create 'locked-in' stresses which will cause cracking to the members. Therefore adequate joints should be detailed within the structure to limit excessive lengths and shrinkage/creep parameters.

9.5.2 Steel

Steel is an efficient structural construction material due to its good strength to weight ratio, providing a ductile material behaving elastically up to its yield strength. Different grades of steel can be produced by altering the chemical composition of the steel material and by keeping the carbon content low, with the addition of other elements in small amounts. Therefore the steel can be produced to suit particular applications, such as reinforcement, piping or rolled sections. These additional elements that vary the material properties include manganese, chromium, molybdenum, nickel and copper.

Therefore, the choice of the steel grade in design should be dependent upon its serviceability requirements during its design life. Requirements such as high strength with fracture toughness, weldability or improved carbon resistance are all achievable through the correct choice of steel grade. The location or intended use of the structure is also a driver of choice.

For serviceable structures in low temperatures, brittle facture is of particular concern and steel strength should be reviewed when choosing the project steel grades.

Failure of steel members is reasonably uncommon as beams along with frame systems are designed to accommodate the required applied loads along with serviceability requirements such as deflections, vibration and temperature. The beam design should include checks for web buckling under high loads, usually found at support positions along with lateral torsional buckling which is dependent upon restraint positions along its length.

There are two particular issues to be investigated when designing in steel, namely, corrosion and fire resistance. Depending upon the location of the structure, the hot strength of steel should be considered along with other protective systems, for instance when designing significant building structures for the power or chemical plant industries where the hot or corrosive environment may affect the structure.

In general though, fire protection of the structure is a requirement of the fire safety regulations and dependent upon the building function, e.g. retail/residential or industrial. The structure is to be designed in accordance with the Building Regulations Approved Document B to ensure that the stability of the structure is maintained for a sufficient time to allow occupants to reach a place of safety.

Steel begins to lose strength in temperatures of around 200°C and then at an increasing rate up to 750°C. Therefore as designers we aim to reduce that temperature gain and strength loss for the required time, by whatever means available. This could be by utilising the inherent protection of a particular system, by increasing section sizes or by adding a passive fire protection system to the structure such as intumescent paint.

The inherent protection available in a structure depends upon the structural system utilised. A steel frame with a slimdek floor system may provide integral protection by protecting the top flange of the floor beam from significant temperature gain for up to one hour without additional protection.

The key to long-term durability of steelwork is the protection of the surface using paint systems to prevent corrosion of the steel, which will eventually lead to section loss and eventually structural integrity.

Within a standard dry building, the protection required is nominal with at least a primer coating applied to the steel following blasting back to bare steel. This can be followed with decorative or intumescent coats, or both, as required.

In extreme external conditions such as highways with regular road salt application or coastal environments with salt-laden atmospheric conditions, the steel protection requirements are more critical. These conditions can lead to faster rates of deterioration and over a period will lead to section loss of the member affecting its load carrying capability. In these circumstances, increased cover to the reinforcement is considered essential.

In the majority of external applications steel is galvanised, i.e., coated with a layer of zinc to protect the bare steel from the elements. This is achieved by dipping the member in a bath of molten zinc. It has been reported that a small number of cases of liquid metal assisted cracking (LMAC) have occurred in the UK whereby the heat of the zinc bath has created a crack along weakened grain boundaries in the structural member. As it is not usual to inspect welds after galvanising, care must be taken when placing steelwork to observe any subsequent breaks in the member surface finish where the ‘hidden’ cracks may open up through the galvanised finish.

The latest protective paint systems for extreme environments now include materials such as thick epoxy paints combined with added glass flake to reduce the solar UV degradation of the paint. In some cases the paint manufacturers have achieved certification of the paint system for providing up to 25 years’ protection before re-application is required.

In such extreme cases the use of stainless steel is also an option, where the steel maintains a protective oxide layer which re-seals if damaged to protect the integrity of the member within an aggressive oxide environment. The use of stainless steel would have to be compared against other protection available, as the material cost is significantly higher than normal structural steel with paint protection.

Stainless steel is also prone to failure by stress corrosion cracking (SCC) which has occurred in a number of swimming pool environments which are found to be aggressive with high temperatures and chemical disinfectants in the atmosphere. SCC may occur where the component is subject to high applied stresses with a susceptible grade of steel in an aggressive atmosphere. It should be noted therefore that this may also apply to other locations such as power/chemical plants where these environments may occur.

Cast iron failure is an area which predominantly occurs in refurbishment of older industrial properties and is discussed later in this chapter. It should be noted, however, that the failure model of overstressed cast iron is often sudden and without warning.

9.5.3 Timber

As structural timber is a cellulosic natural material, it is prone to various types of decay and also insect attack. The type and extent of damage tend to depend upon the type of wood, whether soft or hardwoods and the environmental conditions

Dry rot is the decay of timbers within a building caused by the fungi *Serpula lacrymans* in the UK (**Figure 9.4**). The degradation of the timbers is caused by the fungus and its spreading tubes/threads known as *hyphae* or *mycelium* removing the cellulose and hemicellulose from the timber element. The removal of the timber cellulose leaves behind a brittle timber matrix with deep cuboidal surface cracking, thereby leaving the timber element substantially weakened and prone to failure when loaded.

Figure 9.4 Serpula lacrymans – Dry rot spores © Lutz Weidner. Reproduced under the terms of the GNU Free Documentation License

The fungus can remain active in timber with a moisture content of 20–30% and the threads enable the dry rot to spread across inert material such as brickwork or concrete to reach other timber locations. It is on this basis that treatment of the rot necessitates the removal of all the timbers, plaster and finishes beyond the visible extent of the strands before treating with fungicides. However, it is now thought that removal of the infected timber, eradicating the source of dampness and improving ventilation, is sufficient to prevent the fungus from returning.

As opposed to the dry rot fungi which source moisture in the timber to feed and grow, wet rot is typically the rotting of the timber elements subject to long-term wetting over a period of time with a persistently damp moisture content of 50–60%. This is especially found where the end grain of the timber is exposed in locations such as roof timbers, floor beams and timber joints where the timber will stay wet due to lack of ventilation and gradually rot away over time. The visual sign of wet rot is a darkening (brown rot) or bleaching (white rot) of the timber along with cuboidal surface cracking. Wet rot is responsible for up to 90% of the wood decay in buildings and it attacks both hard and soft woods.

9.5.4 Insect attack

The most common form of wood boring insects in the UK are the Common Furniture Beetle (woodworm) *Anobium punctatum*, the Powderpost Beetle *Lyctus brunneus* and the largest of all, the Death Watch Beetle *Xestobium rufovillosum*.

The Common Furniture Beetle is usually found in the sapwood of both hardwoods and softwoods and usually in damp areas such as timber ground floor construction or roof voids where the timber moisture content is above 12%. The beetles will emerge from the timber usually between May and August and the flight holes will measure 1–2mm in diameter.

The Powderpost Beetle attacks the sapwood of hardwoods such as oak, chestnut, ash and elm, particularly young wood which becomes less susceptible to attack as it ages. Wood beyond the age of 15 years is thought to be immune from further damage from this beetle. The beetles emerge throughout the year with flight holes approximately 1.5 mm in diameter creating fine bore dust.

The Death Watch Beetle is found in large-sectioned hardwood timbers such as oak or elm, where there is already some form of rot present. Therefore this beetle is commonly found in older properties with damp conditions although should the timbers dry out then the attack will slow or even stop. Whilst the oak and elm hardwood is considered the preferred wood of choice by the beetle, the infestation may also attack softwood if close by and/or if rotten. The beetle usually emerges in spring through a 3 mm diameter hole creating gritty bore dust.

As with all these boring insects the larvae feeding within the timber leads to the deterioration, loss of section and overall reduction in strength of the member. Ultimately the section will fail when loads significantly below its original capacity are applied.

9.5.5 Masonry failure

Masonry wall construction is designed to accommodate the transfer of loads vertically down to foundation level and in external conditions, resisting the secondary loading of lateral forces due to wind and temperature.

Concentrating on all masonry construction, we can see that the stability is required in both orthogonal directions and this can be provided for in a number of ways, such as utilising masonry shear cores to transmit the lateral loads at floor levels, utilising the floors as diaphragms or by utilising the internal lateral walls as shear walls buttressing the external walls, thereby transferring the lateral forces to be distributed to ground level.

Critical failures of masonry wall construction are rare and may generally only occur due to lack of care during refurbishment (changing load paths) or by a sudden load condition such as explosion. However, in these cases there are design criteria established in the relevant codes and standards which the engineer should follow to ensure that overall structural stability is maintained. This includes the requirements for 'key elements' in the Building Regulations along with the requirement for horizontal ties at floor levels. In order to restrict lateral deflections of large masonry panels, piers or proprietary windposts are introduced at suitable centres to stiffen the panel and transfer the lateral load to the floor structure.

More commonly, masonry failures may be caused by secondary effects such as thermal and radiation, environmental changes (creep and shrinkage), deflections due to applied loads, ground movement/settlement and vibration effects. As the brick is modular clay with mortar bedding the finish is brittle and any element of movement will be visible in the form of cracking. This cracking can follow either the mortar jointing as shown earlier or can be seen as shear cracking through the masonry unit. As Driscoll and Skinner (2007) point out, 'brick walls are unlikely to have cracked unless there have been centimetres of differential settlement across a typical domestic building.'

The expansion of continuous lengths of brickwork will result in in-plane movement and cracking at wall returns and at wall openings. It has been noted that the potential for cracking at a wall return is greater when the depth of the return is short. The expansion may also account for horizontal sliding on a damp proof course (DPC). Relative movement may also occur between materials of differing thermal coefficients such as concrete and brick or even between block and brick cavity walls. However, in most cases the use of flexible cavity wall ties will accommodate any differential movement between dissimilar leaves.

Care should also be taken where concrete roof or floor slabs bear upon a wall as the expansion of the concrete slab will transmit lateral forces to the masonry leading to vertical cracking in the elevations. Movement joints should be incorporated to accommodate any expected movement, thereby reducing unexpected cracking.

Brick units may also suffer from deterioration due to weather effects over a period of time. Depending upon the original strength of the brick, any inclusions or defects within the clay may lead to cracking thereby allowing moisture to penetrate. Regular wetting and drying along with freeze/thaw action during winter will eventually lead to spalling of the brick face leading to section loss and strength reduction. This does not necessarily lead to failure of the wall but will increase moisture and damp penetration to the inner face of the brickwork.

Wall tie failure is also common in older cavity walls which may be evident by bowing or bulging of the external elevation in certain locations. Where this defect is present at or near a floor level this may also present a failure of lateral restraint, usually provided by the floor construction bearing and tied to the wall.

9.6 Design

Very rarely can building failure be classed as unforeseen. As engineers we are expected to undertake building design with the relevant knowledge gained over many years with our past experiences being disseminated throughout the profession leading to changes in working practices. Critical failures such as the disproportionate collapse at Ronan Point in 1968 led to changes in structural design in practices around the world and it is these practical experiences along with ongoing academic research that lead to changes in the relevant design standards and guides.

Society relies on the protection afforded by the Building Regulations and Codes of Practice and also the engineer experienced in interpolating their requirements. It is the engineer who must exercise engineering judgement to discharge his or her responsibility to provide a stable structure, fit for its intended purpose, that will not endanger its owner, occupants or the public. As IStructE (1990) states, 'The standard of care expected from a structural engineer, in whichever capacity he [or she] is acting, is normally that of a prudent and reasonable engineer, not of one inexperienced in the work he [or she] has undertaken to do'.

This brings us on to the errors that may occur during the design phase of a project including the preparation of structural details. The key to a safe building is the overall stability of the structure. Failure to adequately assess the load paths to ground level, the possible modes of failure and the structural behaviour of the system leads to inadequate provision of restraints and ties necessary to ensure stability and robustness of the structure.

Errors can occur in the interpretation of code requirements, the assessment of loads and also the combination of load cases which may prove more onerous to the design. These errors are exacerbated by failure to recognise or cater for any secondary effects such as residual stresses or fatigue.

Serviceability issues, other than beam deflection, may also be missed. This may lead to inadequate provision for items such as thermal movements/restraints, shrinkage/creep, groundwater or differential settlements.

There may also be a lack of knowledge when specifying construction materials, their use, grades and isolation requirements from other materials. For instance, the instigation of bi-metallic corrosion between steel and stainless steel material requires adequate separation with gaskets and steel has different grades to accommodate the required stress under many different environmental conditions.

Communication between the design engineer and the drawing office is also of paramount importance to ensure that the structural drawings are produced according to the designer's calculations and there is no lack of detail. After all, the drawings communicate the design requirements to the contractor for construction and should therefore be checked to ensure the structure is buildable and that the drawing indicates all the necessary safety issues for the contractor.

Construction stage instability should be considered at the design stage especially if the structural form is elaborate and the design requires special analysis. Any non-standard structural frame that the contractor or subcontract designer (fabricator) may not have experience of should be either annotated fully on drawings or be undertaken by the engineer responsible for the structure, working alongside the contractor to ensure structural safety during construction. Items such as erection sequencing, temporary bracing and connection forces should be detailed, as applicable.

Many errors or altercations at construction stage are due to lack of information or split responsibilities, where one party expects the other to provide additional input, in order to finalise the construction method. Temporary bracing is typical of this, as it usually falls to the steelwork subcontractor. However, as previously mentioned, the engineer responsible for the overall building should ensure that any subcontractor design is compatible with the building design and the construction method of the building.

The engineer should also ensure that the details provided on the construction drawings are thoroughly checked prior to issue. Critical items such as adequate reinforcement, lap lengths anchorages and curtailment, reinforcement congestion, correct material grades (masonry/steel/concrete/timber), embedded items, etc., are all to be reviewed.

One of the most critical areas of construction is that requiring refurbishment of existing buildings. Refurbishment projects can include demolition, foundation improvements, reuse of existing materials and, most importantly, increases in load and changes to the original building load path. In addition, the method of construction may require additional research as such methods have changed considerably over the years. Thick stone walls may be loose rubble filled between the outer stone faces, lintels may be missing with support relying on door/window frames. There may be unbonded brick skins at interconnecting walls or floors may not be tied into walls where expected. The process of structural inspection, part demolition, and removal of existing, possibly load-bearing walls requires careful assessment during the design phase and adequate planning by the engineer.

Stability of the structure is paramount during these works as there have been numerous unplanned collapses over the years. Propping should be planned and installed prior to removal of structural elements thereby ensuring loads are supported and transferred to ground level.

Where load paths are revised – leading to increased loads on existing elements such as walls and structural framing of beams and columns – they should have their capacity assessed for the proposed loads. In older buildings care should be taken to ensure that the steel strength used in assessment calculations is correct with reference to the *Historic Structural Steel Handbook* (Bates, 1984) which provides properties of UK and European cast iron, wrought iron and steel sections and stress data since the mid-nineteenth century. However, in the absence of any existing building information, materials testing may be required in order to assess the material quality and strength characteristics.

Care should also be taken when dealing with brittle materials such as cast iron in refurbishment projects. Cast iron can be present in the form of beams and columns and is regularly found within masonry jack arch construction. In general, many cast iron elements continue to be reliable; however, care should be taken when assessing their residual strength to accommodate a change of loading situation. As with most structural elements, long-standing ingress of water into the construction can lead to corrosion, deterioration of any infill around the beam and ultimately loss of composite action.

It should always be remembered that it is normal for overstressed cast iron beams to fail without warning.

Additional loading on to load-bearing walls and subsequently their foundations requires proper assessment and suitable site investigation. If the existing foundations are found to unsuitable then underpinning in the form of mass concrete to suitable depth or mini-piling may be required to transmit the increased loads to suitable strata.

In all these cases, it is expected that the engineer responsible for the assessment and design of such projects is an experienced, preferably Chartered engineer. Unfortunately where failures have occurred in the past, the work has been undertaken by designers without the proper training or experience to undertake this type of work.

9.7 Analysis

As buildings become more complex with increased heights, layouts and other architectural requirements, the engineering design standards that are used have also developed requiring in-depth analysis. As such, commercial design software has been developed to enable the engineer to both analyse the structure and design the structural elements for both static and, if required, dynamic response quickly and accurately. By establishing a base model, sometimes in conjunction with a building information model (BIM), the design can be revisited and revised accordingly as the client/architect requirements change.

It should be remembered though that these packages are highly advanced computational aids. Some analysis packages are mathematical which require engineering knowledge in order to apply the software to the structural problem, whilst others are structural packages which require direct engineering input.

It is often found that the operators (or analysts) have inadequate experience to either input the required information or assess the output from the software. The user should know how the model deals with criteria such as elastic shortening, member releases and how stresses are distributed between elements. He or she should also have the practical experience and common sense to know that 32 mm diameter reinforcement bars cannot be placed at 35 mm centres as the design software may suggest.

In seismic analysis, the model should (depending on the software) be allocated a damping level when analysing the structure, therefore the operator should have sufficient experience in seismic engineering to understand the requirements and the eventual analysis output. Therefore it is essential that these are not treated simply as a 'black box' by assuming that the output is correct to three decimal places and relying on it for final design. Reality checks should be carried out to give some credence to the output and it should also be an internal/company quality assurance requirement that a verification check is undertaken for each software package in use.

Again, the key to efficient use of complex analysis software is that the experienced engineer knows how the software works, the best way to model the structure and the ability to check and confirm that the data output is as expected.

Whilst the use of finite element analysis (FEA) software is growing and in some cases it is essential when dealing with complex structures, the input and output of any FEA should always be critically reviewed by an experienced engineer. The gravity of such an omission was highlighted in spectacular effect by the failure of the offshore oil platform Sleipner A during ballast testing in the North Sea on 23 August 1991 (Rombach, 2004).

This Condeep Type gravity platform utilised twenty-four cylindrical caisson cells of 24 m diameter for buoyancy during construction and shipping. These cells were located in an array pattern at the base of the structure on the sea bed, from which the four main support legs extended to the surface to support the steel topdeck superstructure (**Figure 9.5**).

Following the accident, investigations revealed a failure of the tri-cell walls during the partial filling of the caissons causing uncontrolled flooding of the caissons and the sinking of the whole structure, leading to a financial loss estimated at 250 million US dollars.

The tri-cell walls were the area of convergence of three caisson cell walls which were found to be subjected to high differential water pressures during the ballast test. However, investigations

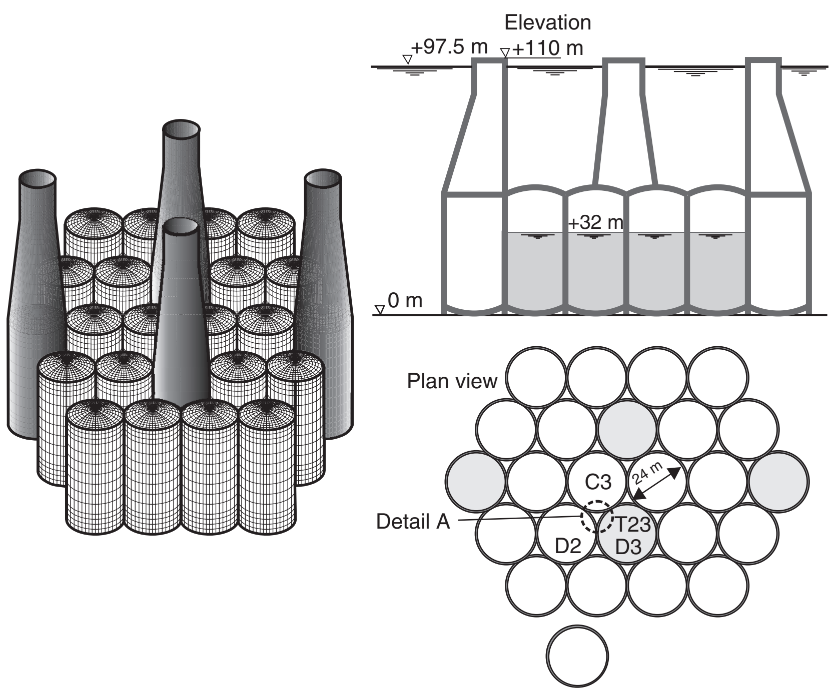

Figure 9.5 Finite element analysis – Sleipner A Oil Platform (Rombach, 2004)

revealed that the failure occurred with a water pressure equal to 67 m water depth, where the structure was designed to operate in 82 m of water. This led to reviews in the analysis and detailing of the reinforced concrete which found that the walls had been modelled using a course FE mesh which was unable to model real characteristics of the loaded shell structure and this led to an underestimation of the tensile forces in the walls by 50%. Also, the detailing of the reinforcement failed to anchor bars across the tri-cell wall into the compression region of the opposite cell walls leading to insufficient reinforcement to prevent shear failure at the joint (**Figure 9.6**).

Detailed knowledge of FEA and material behaviour is thus essential when analysing structures and where possible critical areas should always be analysed separately using alternative models or refined FE models. This ensures that detailed loading effects and forces are catered for in the design.

Here, the detailing of structures has been shown to be a contributing factor in the failure of the structure. Structural detailing requires experience and training both by the drafter and the checking engineer to ensure that the drawings adequately communicate the design intentions whilst being 'buildable' for an experienced contractor. The overall safety of the structure should be under the supervision of the most experienced engineer and in the United States this is the Engineer of Record (EoR) who signs off the design and construction drawings. However, errors can also occur with this system especially when elements of design are changed or become the contracted responsibility of others, such as connection design by the fabricator.

This was a key issue in the Hyatt walkway collapse in Kansas City, 1980, where a catalogue of revisions, missing detail information on drawings and fabricator changes to the hanger support led to the aerial walkways being constructed without being adequately designed. It was later found that the connection details were not sufficient to support the service loadings under the local building design code (Delatte, 2009) let alone the increased loads at the hotel function that night. Unfortunately, 114 people died and 200 people injured when two of the aerial walkways collapsed in the hotel lobby leading to billions of dollars in damages awarded to the victims, their families and rescuers.

This highlights the issues regarding the responsibilities of the whole construction team involved in any design, regardless of their contractual responsibilities, to ensure that the materials used, the design of the structural elements and the overall structural system are all fit for purpose.

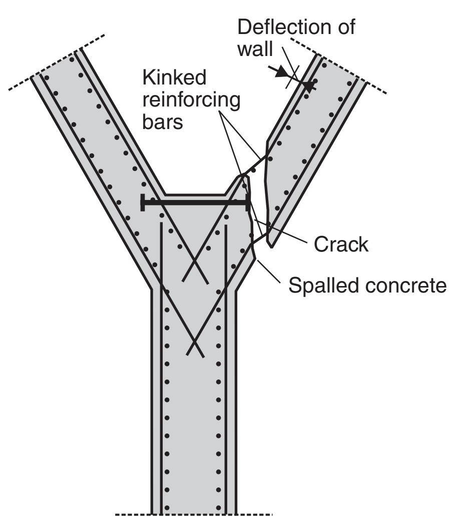

Figure 9.6 Reinforced concrete failure – Sleipner A Oil Platform (Rombach, 2004)

9.8 Conclusion

There are many mechanisms of building failure that need to be considered, from the initial scheme design and material selection through to analysis, detailed design and construction completion.

Consideration has also to be given to the serviceability issues which may, over time, lead to degradation of the structure and a subsequent reduction of the structural capacity leading to failure.

Most importantly, the responsibility for the design, the construction stability and the serviceability of the structure should be allocated to an experienced and qualified engineer to ensure that the design and materials are adequate for inclusion in the permanent works.

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9.9.2 Useful websites

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Confidential Reporting on Structural Safety: www.cross-structural-safety.org

Construction Industry Research and Information Association: www.ciria.org

Health and Safety Executive: www.hse.gov.uk

Institution of Structural Engineers: www.istructe.org

Standing Committee on Structural Safety: www.scoss.org.uk

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Chapter 10
Loading

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This chapter covers loading from the context of practising engineers engaged in the design of structures. The different types of loads (dead, live, wind, etc.) are categorised and the modes of application are explained. Permanent (dead) loads are explained and examples of values given. Imposed (live) loads are covered, explaining the different occupancy classes and guidelines particularly for preliminary design. Wind loads are discussed and rules of thumb given. Earthquake (seismic) loads are discussed along with key considerations for their consideration in the design process. Blast loads are described, and guidance given on how to calculate them and their effect on the structure. Self-straining load effects (temperature, shrinkage, movement, creep, parasitic 'loads' due to pre-stressing) are described. Fire, fluid, silo and ground/soil loads are discussed and guidance given on how to approach them.

CONTENTS

10.1 Introduction

One of the critical variables affecting the true factor of safety in the design of structures is the accuracy of the magnitude and method of application of loads. Given that much of the analytical work which is carried out in practice now uses computer programmes with a high degree of precision it should be apparent that the 'accuracy' of the results depends greatly upon the accuracy of the loading – if the loading is either over- or underestimated then the precision of the analysis does nothing to improve the overall 'accuracy' of the analysis.

It will generally be possible to estimate with a reasonable degree of accuracy the magnitude and distribution of the permanent loads associated with the structural self-weight and applied finishes; however, it is important that during the course of the design process these are checked and refined a number of times, as often architectural and other changes will occur which influence the loading. Live loads are by their nature harder to quantify both in terms of magnitude and distribution and their variability in time, hence these are often dealt with based on an empirical approach (i.e. based on codes, which in turn are based on traditional practice, rather than being derived from theory).

Significant research has gone into the development of approaches for determining more complex load effects, such as wind loading, seismic loads, the various 'self-straining load effects' such as temperature and shrinkage and more recently complex 'load' effects such as blast and fire loads.

Intrinsically, all loads are in fact variable in time, both in magnitude and distribution and the distinctions we derive relate to establishing a workable theoretical model of load behaviour as part of the overall structural idealisation. Hence, derivation of loading is a matter for engineering judgement, rather than application of codes or standards.

In terms of what is 'reasonable' it is important to understand that particular types of structural design (say for a water tank, or a lightweight portal framed industrial building) will generally have structural engineers who specialise in their design and it is generally useful to study references related to particular usage if designing such structures for the first time.

10.2 Typology and method of application

Given the wide variability in load effects on structures there are a number of different ways in which these loads can be grouped for the purposes of understanding their similarities and differences. Some groupings are as follows:

a. Permanent mass related loads
   i. Self-weight of the structure
   ii. Superimposed dead loads (finishes)
   iii. Static mass of fixed equipment
   iv. Weight of soil supported

b. Movable (time-dependent) service loads
   i. Distributed floor loads
   ii. Concentrated live loads (traffic or movable equipment)
   iii. Loads from stored materials (grain silo for example)
   iv. Loads from stored liquids

c. Environmental forces
   i. Wind loads
   ii. Flowing or ponding water loads
   iii. Snow loads

d. Self-straining forces
   i. Temperature
   ii. Concrete shrinkage
   iii. Differential movements of supports

e. Inertial forces

  i. Dynamic wind loads
  ii. Seismic loads
  iii. Blast loading

Time variable loads can be grouped as:

a. Load effects expected during typical 'service' – general live loads say
b. Rare event loads (extreme events, such as floods, say)
c. Extremely rare loads, i.e. worst case loads

Conceptually loads can be grouped based upon their time dependency;

a. Long-term (permanent) loads – expected to remain static for expected life of structure
b. Short-term, fixed magnitude events
c. Constant or random variability fluctuating forces
d. Dynamic loads (impact, seismic, blast loading)

For load effects which are variable it is important to consider the impact of a variable distribution – the simplest example of this is for a multiple span beam, whether the loading case of maximum live load on one span with zero live load on adjacent spans as well as the inverse to capture the peak moments at the mid-span and supports, as well as potentially a point load adjacent to a support and a uniformly distributed load. For more complex structures (say a 3D canopy structure) the impact of spatial distribution of time-dependent loading is more difficult to capture, although analysis methods have been developed that give less conservative loads than would otherwise need to be used.

Generally dead and live loads will be applied as uniformly distributed loads (UDLs) and point loads, depending upon whether the idealisation of the structure is two- or three-dimensional the loads will be either area, line or point load effects.

Wind loads are often applied as UDLs on the surfaces of a building, or by application of point loads at the centre of projected area – however, it should be considered that for some cases (tall buildings for example) the dynamic component of wind loads (i.e. that portion due to the movement of the structure under the wind) is significant and those effects are distributed around the centre of mass, rather than the centre of projected area.

Seismic loads are simplistically applied as equivalent static loads, which are typically taken to be a first mode (i.e. half curve) response which gives an 'inverted triangle' of forces typically applied at the centre of mass (or more accurately applied eccentrically around the centre of mass in order to generate a level of additional torsion to reflect the variability of mass distribution). More sophisticated dynamic analysis procedures (e.g. response spectrum analysis or time history analysis) are carried out, either due to the level of risk (i.e. in areas of high seismicity) or due to the judgement being taken that the static method does not reasonably reflect the behaviour of the structure (for example where the structure is highly irregular and hence the assumption of a principal translational mode of behaviour is not applicable).

Self-straining load effects are typically modelled as either 'equivalent temperature' effects (e.g. for shrinkage the strains would be calculated and then converted to an 'equivalent' temperature change) or support movements, and analysed using an elastic computer programme – it should be noted that these sorts of load effects are very dependent upon the details of the analysis and more prone to give conservative results (particularly around boundaries where infinitely stiff translational or rotational restraints have been modelled).

Blast loads must be modelled accounting for the dynamic effects. This is typically considered using dynamic 'single degree of freedom' methods. It is often possible to derive simple equivalent static UDLs (Cormie, 2009), or it can sometimes be more efficient to design the structure via an assessment of the local effects on the remaining structure (i.e. key element removal). More sophisticated dynamic time history analysis using nonlinear finite element analysis is sometimes carried out where simpler methods are not suitable.

Fire loads are typically modelled by varying the combination factors, rather than by changing the actual magnitude or distribution of the gravity effects.

Fluid loads are typically modelled as variable magnitude pressures (UDLs), based upon the height of retained fluid and its density.

Silo loads are modelled in a similar fashion to fluid loads, but with generally with more complex variation (to reflect the 'non-fluid' behaviour of a 'grain', for example, as well as the influence of say 'emptying' a silo.

Soil/earth loads are typically applied as constant UDLs at the underside of a structure and linearly varying lateral loads on the sides (but with surcharge loads being applied as uniform loads).

10.3 Combinations of load

The Eurocodes and the various US codes follow roughly similar approaches of using factored load effects for the ultimate (or strength in US terminology) limit state (ULS) and unfactored loads at the serviceability limit state (SLS).

All combinations are related to the likelihood of combinations occurring; it should be self-evident that applying simultaneously wind, live, temperature and seismic loads effects, whilst logically sustainable to variable relative levels, is not a practical approach.

Engineering practice is therefore to define a number of typical combinations, such as:

Dead + Live

Dead + Live + Wind

What is often less clear is how to combine the extreme event load effects (for example, temperature). These are typically added together as 'extreme' events, which means a lower 'combination' factor for the dead and live components recognising the lower probability of both an extreme event and high permanent and imposed loading happening simultaneously.

10.4 Permanent (dead) loads

The self-weight of a structure is design-dependent and tends to vary over the course of the design process (generally in a reducing trend). During a typical design process the accuracy of the estimate of self-weight can be improved by considering:

- The longer the span – the more important the accuracy of the self-weight.
- Depending upon the nature of the load combination being considered, overestimating the dead load can either be conservative (i.e. say with a simply spanning beam) or non-conservative (i.e. when considering a lightweight steel roof structure under a wind uplift condition).

Typical densities used to determine dead loads are shown in **Table 10.1**.

At the concept design stage it will generally be required that the engineer will assume finishes loads. Typical values used would be:

Hard floor finishes (screed, tiles, stone)	20 kN/m^3
Raised access floor	0.5 kN/m^2
Façade (curtain wall)	1.0 kN/m^2
Suspended services (office)	0.25 kN/m^2
Ceiling/lighting	0.25 kN/m^2
Block partitions	4.5 kN/m^2
Car parking – allowance for kerbs, etc.	0.5 kN/m^2

10.5 Imposed (live) loads

Live, imposed or service loads are variable loads which occur due to the intended purpose or use of the structure being designed. By their nature they are less definable than gravity loads determined based upon what is built. They are also subject to being changed as the building use changes. It is generally accepted though that new-build structures will be designed for the use currently intended and that the design load will be recorded so that any future change in use can be considered based upon the loads of the new use and the actual capacity of the structure.

For typical building uses there are code tables which provide values to be used for design of the structures. These are not repeated here and are generally suitable for most design uses.

Consideration should be made to the use of live load reduction for the design of individual members depending upon the tributary areas.

Material	Density
Concrete (normal weight):	24 kN/m^3
Concrete (lightweight):	18 kN/m^3
Steel:	77 kN/m^3
Concrete block density:	20 kN/m^3
Lightweight concrete block work (for internal uses):	7 kN/m^3
Soil for planters (saturated):	20 kN/m^3
Screed:	20 kN/m^3

a. Self-weight – including rules of thumb and typical values
b. Superimposed dead loads – typical values and densities/weights for typical build-up of finishes, etc., material densities

Table 10.1 Typical densities used to determine dead loads

10.5.1 Service (occupancy) loads – typical values (and why)

Generally engineers would allow for partition loads of 1.0 kN/m^2 as LL for an office building (combined with a 2.5 kN/m^2 occupancy load to give 3.5 kN/m^2).

Rules of thumb:

- Always mark up a set of current architectural plans with the loadings used – colour coded/hatched to differentiate areas of different usage.
- Always double check that the 'apparent' use of spaces is understood (i.e. discuss with the architect/client to understand the usage).
- Design assuming any atriums/voids will be 'filled in' later and for the building usage to be extended over that area.
- Design any flat roofs which are accessible for crowd loading (5 kN/m^2).
- Non-trafficable roofs should be designed for service loads due to stacking of roofing material and equipment for maintenance purposes.
- For very large roofs (exhibition halls, etc.) it is not appropriate to use the code specified 'minimum' live load – a specific analysis of reasonable anticipated loading should be carried out and the relevant 'operations and maintenance' manuals updated with the actual loads designed for.
- For roof trusses and rafters allow for a 5 kN point load at a single bottom chord node (i.e. consider next to support and then in the middle to cover for maximum shear and maximum bending moment cases, or in the appropriate location to generate the worst forces being checked).
- For trusses deeper than 1.2 m allow for a 1.2 kN point load applied mid-way between nodes (without other service load effects).
- Generally design offices for 3.5–2.5 kN/m^2 for service use and 1 kN/m^2 for lightweight partitions. Do not allow for higher filing loads unless there is a particular requirement for high-density filing (current trends are away from a lot of heavy equipment and storage in offices).
- Avoid complicated mixing of live loads – if in doubt use a higher blanket value as the risk of design changes making your initial loading distribution inaccurate is high.

- For areas with crowd loading consider carefully whether vibration will be an issue.

10.5.2 Plant and equipment loads – typical values (and why)

In order for buildings to operate they require sometimes significant amounts of building services plant; there are also structures which support plant and equipment involved in servicing another building structure, or which are related to industrial processes.

Generally the code recommends a loading minimum of 7.5 kN/m^2, which is supposed to be checked against 'actual' loads. Obviously at the early stages of a design process it is unlikely that the 'approximate' weights of plant will be available and it is generally the case that even at the point of 'completion' of the design the actual final equipment load will be known.

Using 7.5 kN/m^2 for a typical building (office, residential, etc.) for any plant areas is generally conservative and appropriate; however, for tall buildings and areas with large plant areas a value of 10, 12.5 or 15 kN/m^2 may be more appropriate.

It should be recognised that a lot of equipment supported by structures will contain oscillating parts which will impose dynamic loads greater than the 'at rest' weight of the equipment. It is also important to not use 'shipping weights' but actual 'operating weights' particularly for items of plant containing liquids of various sorts (principally water in the context of buildings).

Rules of thumb:

- Allow 20% above 'operating weight' for typical elements of plant.
- Anything which involves hoists or winches (lift machinery) should be designed for 100% of the 'dead weight' of the load – including weight of cables, tackles, etc.
- For tall buildings and for 'energy centres' allow a minimum of 10 kN/m^2 and consider water tanks and cooling towers in detail for their weights.

10.5.3 Crane loads – tower cranes (foundations), workshop cranes, etc.

Cranes are divided between those (typically) larger cranes used during construction (tower cranes and 'crawling' or mobile cranes) and workshop or building integrated cranes.

The design of tower cranes and mobile cranes for use in construction is done to particular codes of practice and standards applicable to the location. In general, the structural designer will be provided with reactions from the crane manufacturer which will provide 'service' level loads for the crane for use. It is not general practice to allow for construction loads on the permanent structure as a building designer – typically the contractor will check and strengthen the structure if required. It is often more important to understand the way in which cranes will be supported and in the case of a tall building which has a tower crane climbing within the core during construction it is one of the factors to consider in the design of the core geometry.

The type of crane which is probably most often encountered by building structural engineers is a runway girder supported workshop crane; these vary in size from effectively simple manual hoists to very significant cranes in steel fabrication yards for example. Typically, the structural designer will design the runway beam and then the primary structure to which the runway beam attaches. It is important to understand that the crane will impose not only vertical loads but also transverse and longitudinal loads (due to dynamic effects, braking, etc.)

Rules of thumb:

- Use a dynamic amplification factor of 100% over the rated 'lifting capacity'.
- If you are not sure on the capacity assume 10 metric tonnes and look at a crane manufacturer's literature and clearly state your assumptions.
- Allow a 10% lateral load at each wheel location of a runway crane.

10.5.4 Vehicle loads – highway, fire truck, garbage truck, cars, impact loads

For general building structures the ramps and loading bays contained within need to be designed for the largest wheel loads likely to occur in service.

For car parking structures the recommendations of the ICE/IStructE guide to the design of parking structures can be followed as a starting point.

One significant loading effect is vehicle impact. To assess what sort of loading is suitable it is necessary to understand the type of vehicles which will be used and also to consider the type of protection (kerbs, bollards, etc.) which are provided to the structure.

Rules of thumb:

- For a 'car parking' area only – allow for 2.5 kN/m^2 UDL and 10 kN point load.
- For a 'general commercial vehicles area' – allow for 5 kN/m^2 UDL and 35 kN point load.
- For a 'heavy highway vehicle' – allow for 10 kN/m^2 UDL and 50 kN point loads – at 1.2 m × 1.8 m pitch.
- Braking forces – allow 10% of the vehicle mass in any horizontal direction (i.e. in worst case direction).
- Vehicle impact loads:
  - For columns/walls near a driveway, without massive bollards or rails – allow 200 kN at 1 m above kerb.
  - On columns/walls protected by a robust structure – allow 100 kN at 1 m above kerb.
  - On rails, etc. designed to protect the structure or prevent fall of the vehicle – allow 100 kN at 0.5 m above kerb.

10.6 Wind loads

Wind loads are one of the most significant lateral loads considered in the design of structures, since they are generally considered in the design of building structures, compared to say seismic loads, impact of blast, etc. which are considered in some cases depending upon location and 'risk'. The magnitude of the loads is dependent upon the geographical location, the statistical frequency of storms (climatic analysis), the roughness of surrounding ground, the geometrical features of the building and its parts and the dynamic properties of the structure. Additionally, the magnitude of loading considered to larger building volumes is less than that to smaller areas, such as individual cladding panels, as the gusts are larger, with less speed and correlation of wind over larger volumes is less probable than over smaller areas.

Wind loads for typical shapes of buildings and other structures are generally determined simply through use of the various code approaches – BS6399, EC3, IBC (ANSI ASTM), etc. In recent years, there has been a trend towards these codes being more consistent and a consensus of approach between European and North American practice has developed.

Some rules of thumb which can be used for initial hand calculations are as follows:

- lateral load of 1.5% of dead load;
- 1 kN/m^2 pressure for roof structures, etc.;
- 2 kN/m^2 pressure for tall buildings;
- values for cladding pressures double those for buildings (i.e. 2 or 4 kN/m^2 for design of a cladding element or supporting member).

Whilst wind loading may govern some aspects of many, if not all building types, types of structures for which wind loading is likely to be particularly significant are:

i. tall, slender structures;
ii. long span roofs;
iii. local areas in buildings impacted by funnelling effects (i.e. two buildings close together, etc.).

Keep in mind that wind will hit tall structures at high level and be forced down and around, so the local pressures can be significantly higher than at that height for a different structure

Wind loads can be thought of as having two components: the static part which is due to the shape of the building and the dynamic part which is due to the building's response to the wind loads and which is dependent upon the mass and stiffness distribution of the structure and its intrinsic (or augmented) damping. For common structures, the dynamic part of the loading can be derived by applying simple factors to the static part or by using other codified methods.

To deal with more sophisticated cases the currently available techniques are either computational modelling or wind tunnel testing. Wind tunnel testing seems to be universally adopted, due mainly to speed and cost considerations as well as questions regarding the 'accuracy' of computational methods. When using wind tunnel testing it should be recognised that in the vast majority of cases only the static component is directly 'measured' and the dynamic component is added numerically, which in particular for torsional modes effects is not 'precise' – i.e. even apparently 'precise' results do contain considerable approximation and need to be understood in such terms.

For tall buildings typically the guidelines for use of wind tunnel testing are as follows:

i. has a shape which is not a typical geometrical form (i.e. it is not a box);
ii. natural period higher than 1 second;
iii. subject to buffeting by wake of upwind structures;
iv. subject to funnelling effects due to ground conditions or other structures.

For structures supporting large roof areas it is often possible to generate 'savings' in terms of wind loadings by using wind tunnel modelling which looks at the correlation of wind effects over the whole surface compared to code approaches.

It should be borne in mind that typically something like 25% of structures which are wind tunnel tested give wind loads that are higher than in the code approach.

10.7 Seismic loads

Seismic 'loads' are those forces generated by the accelerations related to earthquakes. Whilst the design of structures for seismic loads has not been commonly undertaken in the UK (with the notable exception of the nuclear industry) it is an important part of structural engineering design globally, in particular in those countries where earthquakes of significant magnitude to cause damage to property and injuries and/or death of people occur regularly.

The most commonly adopted international codes are those in the UBC 'family' (UBC97, IBC 2006, etc.), which are US codes. The Eurocode approach (EC8) is similar to the US codes in theoretical approach and application. The Japanese seismic codes are significantly different in approach, being based on a 'strength' rather than ductility approach. In recent years there has been a trend towards performance-based design methods and more sophisticated dynamic methods of analysis, reflecting the general changes in engineering computational methods.

When considering the magnitude of seismic loads it is important to start with an understanding of what is being designed for. Typically the approach is:

- Structures to survive minor, regularly occurring earthquakes without permanent damage and associated non-structural elements to have no, or minor, damage.
- Structures to behave as predicted for larger magnitude, infrequent earthquakes – extensive damage to the non-structural elements is acceptable.

- Under very high magnitude and very infrequent earthquakes for the structures to not collapse (i.e. failure modes which are not 'brittle' and which allow the evacuation of the structure).

The magnitude of the forces which are developed within structures is primarily governed by the peak ground acceleration (PGA) designed for – the PGA is derived by a statistical analysis of earthquakes and the attenuation of the associated ground motions for the location under consideration, plus an allowance for local soil effects (classically the example of the Mexico City earthquake where many structures are located on deep clay layers, which amplified the earthquake induced ground accelerations experienced by the buildings affected). The commonly used measure of risk is a 'rock' PGA with a 10% probability of exceedance in 50 years, which corresponds to a 475 year 'return period' and maps of 'seismic hazard' are typically prepared on that basis. For the 475 year return period locations can be consider to fall into different 'categories' as follows:

- PGA approximately 0.075 g – Low seismic risk – use of 'equivalent static methods'.
- PGA of approximately 0.15 to 0.20 g – Intermediate seismic risk – detailing and some limits on design.
- PGA of 0.3 to 0.4 g – High seismic risk – requires consideration of dynamic analysis procedures, special detailing, limits of 'types' of lateral load resisting systems and in general particular attention from the conceptual level.

Typically it would be acceptable for engineers without a background in seismic design to undertake designs in areas of low seismic risk and with advice/support to carry out design in areas of intermediate seismic risk. It would be considered unadvisable for engineers who have not been appropriately trained and who do not have sufficient experience, to design structures within an area of high seismic risk. The possibility of errors within the concept, let alone within the detailing, is sufficient to make recourse to an 'expert' advisable.

The natural frequency, or more frequently the fundamental period (i.e. the inverse of the natural frequency), of a structure is typically significant. This is the period which corresponds to the 'first mode' dynamic response, which can be thought of as a 'half sine wave' – this gives a linear acceleration with height (i.e. the 'inverted triangle') of forces typically assumed by engineers designing for 'seismic loads'.

Rules of thumb for estimation of building period:

- $T = 0.56\sqrt{(EI/mL^4)}$ – where m is the equivalent distributed mass, E is the composite Young's modulus and I the second moment of inertia, with L being the height of the 'cantilever'.
- $T = C_t(h_n)^{3/4}$ – where C_t is a coefficient that varies depending upon the type of lateral load resisting system and h_n is the overall height of the building.
- $T = n/10$ – where n is the 'number of storeys'.

It should be noted that these rules of thumb have a high degree of inaccuracy (± 50%) since the dynamic behaviour of real structures is so variable. It should also be noted that for example the second method shown above will generally give lower values than would be derived from a 3D structural analysis of a structure and some codes limit the use of 'too high' a period to a proportion of a 'code derived' value, so that the engineer does not suffer from 'sharp pencil' syndrome.

Some key aspects of seismic loads are as follows:

a. Ductility – a basic principle used in most seismic design codes is that buildings are designed for 'equivalent forces' – effectively scaled inertial forces. This allows for dissipation of energy through structural and non-structural mechanisms (for example, the formation of 'plastic hinges' in a reinforced concrete beam).
b. Capacity Design – another basic principle, which is to design a structure with elements sized and detailed to 'force' a ductile, rather than 'non-ductile' failure mechanism – the classic example being the 'strong column – weak beam' approach, where in a moment resisting frame the strength of the column is set relative to the beam so that a plastic hinge will always form in the beam rather than in the column (and hence avoidance of a collapse mechanism).
c. Design Philosophy – The general approach when designing for high seismic loads is to develop a clear lateral load resisting system, which preferably should be symmetrical on plan and with the centre of stiffness aligned with the centre of mass (to reduce potentially problematic torsional effects) and to avoid sudden changes in stiffness or strength vertically (particularly soft or weak storeys in multi-storey buildings.

Generally seismic design codes assume that base shear is determined by a formula such as:

$$V = \frac{CIW}{RT}$$

C = Coefficient based on the PGA (peak ground acceleration) and soil conditions

I = Importance factor

R = Load reduction (ductility) factor

T = period – it should be noted that codes generally contain approaches for approximating periods for different structures or else more sophisticated numeric analysis techniques can be used – such as modal analyses of 3D FE models; codes often contain 'limits', recognising that any analysis, no matter how apparently precise, is in fact by nature relatively inaccurate

W = seismic weight

For example, for a 10-storey office building, with a reinforced concrete shear wall system the period might be 1 second and in an area of high seismic activity on hard rock the base shear would be

$$V = \frac{(0.32)(1.0)W}{(5.5)(1.0)}$$

$$= 0.058\ W \text{ (i.e. 5.8\% of building weight)}$$

The important point to consider with the example above is that the 'elastic' inertial forces would be 5.5 times as high (i.e. 32% of building weight); in order to justify the reduction it is necessary that the detailing of the structure be such that the ductility is assured.

The loads are distributed in their simplest form as an 'inverted triangle', via a formula such as:

$$F_x = \frac{(V - F_t)w_x h_x}{\sum w_i h_i}$$

where

V = Seismic base shear

F_t = a concentrated load at the top of the building to allow for 'higher mode' effects

w = floor weight

h = height above datum

This distribution of force effectively assumes a predominant first translational mode (i.e. a half wave shape).

For example for the 10-storey building mentioned above, assuming

W1 to W10 = 10 000 kN

V = 580 kN

F_t = 0 (say)

Level	Floor force: kN
10	105
9	95
8	84
7	74
6	63
5	53
4	42
3	32
2	21
1	11

For more critical buildings and in particular where 'unusual' features such as soft storeys, major mass and stiffness asymmetry, etc. are part of the design and in areas with very high risk then more sophisticated analytical methods (dynamic methods) are used – the main ones are response spectrum analyses and time history analyses. Both require specialist software and knowledge to undertake (particularly a time history analysis) and it would not be advisable to carry out these forms of analysis without reference to someone with experience in such work (i.e. they should not be undertaken based on reading the code requirements alone).

10.8 Blast loads

Blast loads on structures can arise due to gas explosions, dust explosions, the explosion of a pressurised vessel, detonation of unexploded ordnance, accidental detonation of stored munitions/explosives or deliberate (i.e. terrorist) explosive attack. Other than the explosion of a pressurised vessel, which is a 'mechanical' explosion defined as the sudden release of stored pressure energy, all involve a rapid chemical combustion of the explosive compound. The high-temperature high-pressure products of combustion are initially in disequilibrium from the surrounding air, and hence a transient blast wave is produced which propagates away from the source of the explosion. The peak pressure (sometimes termed 'overpressure' to denote the pressure measured relative to ambient) decreases exponentially as the blast wave expands. The propagation of the blast wave is sufficiently rapid that the momentum causes the air to expand below ambient pressure and a negative (i.e. below-ambient) phase to be set up as equilibrium in the air is gradually restored (**Figure 10.1**).

When the blast wave impinges on an object it is reflected from it, producing a blast load which can significantly exceed the 'incident' pressure observed when the blast wave is propagating in free air. For vapour cloud and dust explosions this 'reflection factor' is approximately 2.0 for surfaces normal to the direction of the blast wave as the flow may be considered incompressible. For high explosives, the reflection factor can be much higher, in excess of 12.0 or more when the pressure gradients are very steep close to the source of the explosion. The reflected blast pressure is the pressure observed by the structure, and also varies with the angle of incidence of the blast wave. It is maximum when the surface is normal to the direction of propagation of the blast wave, and decreases to unity when the blast wave propagates parallel to the surface. For this reason, the incident pressure is also sometimes known as the 'side-on' pressure.

In the design of buildings in petrochemical and process plant facilities, the design case is usually the vapour cloud explosion. The blast wave propagates as the combustion progresses and the rate of combustion defines the strength of the blast wave. The blast load calculations for vapour cloud explosions often quote the incident or side-on pressure, leaving the structural engineer to calculate and apply the reflection factor to give the reflected pressure. Dust explosions are similar to vapour cloud explosions; explosive dusts can include sawdust, animal feeds, flour, coal and so on.

A high explosive is classified by a detonation, meaning the explosive shock front propagates supersonically through the explosive material. The explosion sets up a blast wave which then propagates into the surrounding air. High explosives are

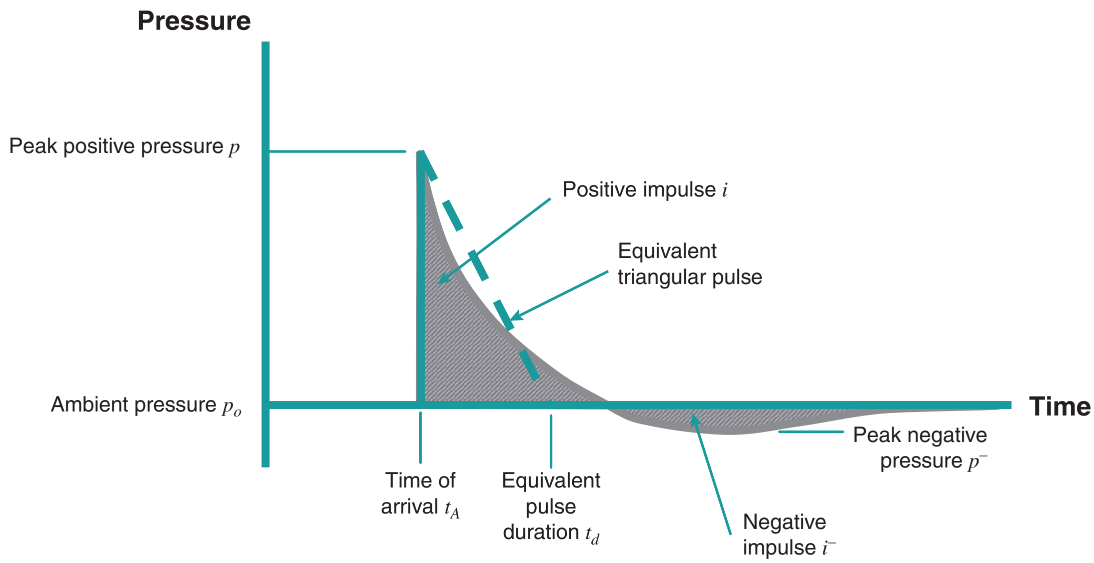

Figure 10.1 Typical blast waveform from a high explosive and equivalent triangular pulse

usually considered for design purposes as an equivalent quantity of trinitrotoluene (TNT), the standard 'reference' explosive on which empirical load curves are based (US Army Corps of Engineers, 2002). For convenience, both incident and reflected blast wave parameters are usually expressed in terms of a scaled distance Z, defined as the ratio of the stand-off distance from the centre of a spherical charge to the target and the cube-root of the charge mass expressed as an equivalent quantity of TNT. This cube-root scaling law holds that the incident and reflected pressure are proportional to the cube-root of charge mass, permitting the blast wave parameters to be plotted as variables which are independent from the mass of the charge. Detonation in materials other than air, such as detonation in soil and underwater detonations are beyond the scope of these relationships and specialist advice should be sought. Similarly, detonations in enclosed volumes (i.e. internal rooms) and the confinement effects of objects such as street canyons or reflection from nearby objects which prevent the free expansion of the blast wave from a detonation in open air can significantly exacerbate the blast loads and require specialist methods of analysis.

Unfortunately, and unlike vehicle impact, seismic and other load types, no simple empirical formulae or 'rules of thumb' are available for the estimation of blast loads and recourse is necessary to these curves to derive loads for design. Recourse is sometimes suggested to the 34 kPa pressure recommended for design of elements designated as key elements for design against disproportionate collapse (see Chapter 12: *Structural robustness*), but this is a notional, static, load for the notional enhancement of the structural design of critical structural elements and it is unrelated to the blast pressures derived by an explosion; therefore the blast pressures due to the event being considered must be explicitly calculated.

For structural design, the engineer will need to know both the peak reflected pressure and the reflected impulse of the blast wave at the relevant scaled distance. The impulse is the integral of the pressure wave over the duration of the positive phase (**Figure 10.1**). It is common practice to neglect the negative phase of the blast wave in most circumstances. The peak pressure and impulse are the two fundamental loading parameters. The shape of the blast pulse is also important, and for high explosives the pressure rise is almost instantaneous followed by a more gradual decay, often approximated by a linear falling triangular pulse of equal impulse to give an equivalent pulse duration td (**Figure 10.1**). For vapour cloud explosions, the 'rise time' is more gradual, and the exact shape will depend on the strength of the explosion.

The magnitude of blast pressures can be significant: for high explosives the peak pressures can be in the tens of megapascals (thousands of psi); however, the duration of the pressure pulse may be only a few milliseconds. It is imperative that the structural engineer design the structure dynamically: single degree of freedom techniques are industry-standard and the approach is described in Cormie (2009). The relationship between the peak pressure and the impulse will govern the dynamic response of the structure. The dynamic response is characterised through the ratio of the blast pulse duration td to either the natural period of the structural element T, or the time to maximum response of the element tm. If $t_d/T > 10$ or $t_d/t_m > 3$, the structural response is quasi-static and the peak pressure may be straightforwardly applied as a static load. If $t_d/T < 0.1$ or $t_d/t_m < 0.3$, the load has decayed before the structure has started to respond and the structural response is impulsive, being largely governed by its inertia. Between these limits the structural response is dynamic, being a function of the inertia and the resistance of the system and defined by the dynamic equation of motion

$$F(t) = m\ddot{y}(t) + ky(t)$$

where, neglecting damping,

$F(t)$ is the time-varying applied load

$y(t)$ and $\ddot{y}(t)$ are, respectively, the displacement and acceleration of the system varying with time

m is the mass of the system

k is the resistance of the system.

Blast loads should usually be applied in combination with the unfactored dead load and a reduced live load (typically a partial load factor of 0.5 is applicable for the imposed load). It is important to define performance criteria for the system, corresponding to the performance criteria described for seismic loads above. For elastic design, dynamic load factors (DLFs) may be calculated (Biggs, 1964) which give the equivalent static load based on the t_d/T ratio for the system and applied load and allow static methods of design to be adopted.

However, explosions are at worst an accidental loadcase and are sometimes a malicious event, and therefore the required level of performance is often limited merely to the avoidance of gross structural collapse. Designing the element to remain elastic under the blast load is usually grossly inefficient and uneconomic, and for an efficient design a degree of plasticity should be permitted to develop in the system. However, where plasticity is permitted there is no theoretical equivalent static load. Nevertheless, chart-based solutions are available for systems (Cormie, 2009) which incorporate plasticity without recourse to numerical time-stepping solutions of the equation of motion. Provided the permanent deformation is limited such that any requirement for residual load-bearing capacity (which may involve second-order effects due to the lateral deformation of columns) is maintained, this approach is usually acceptable, and guidance is available (Cormie, 2009) on the limiting values of ductility and deformation permitted in design.

Rules of thumb:

- Elastic design will always be conservative, but will usually lead to a highly inefficient and uneconomic design.
- For shock-fronted loads, the worst-case dynamic load factor for elastic design occurs where $t_d/T > 10$ (quasi-static), where the DLF tends to 2.0.
- For isosceles loads (typically associated with hydrocarbon vapour cloud explosions), the worst-case dynamic load factor for elastic design occurs where the pulse duration is approximately equal to the natural period of the system ($t_d/T \approx 1.0$), where the DLF $\approx$ 1.6.
- At the impulsive limit ($t_d << T$), the dynamic load factor for elastic design tends to zero.
- Allowing plasticity in the system will usually result in a significantly more efficient design.
- Design of reinforced concrete will usually be limited by the end rotation of the element and will often be shear-critical.
- Design of structural steel will usually be governed by the flexural response and limited by the allowable ductility.

The flexural and shear forces due to the direct application of the blast pressure are accompanied by additional failure mechanisms due to the internal stress waves set up within the material when the blast wave impinges upon and propagates through it. The brittle failure that can result is a particular issue in reinforced concrete, but at small stand-off distances can also cause the brittle failure of structural steel and other ductile materials.

10.9 Self-straining load effects

Self-straining loads, or more accurately self-straining forces, can occur with or without the physical application of external load.

The application of a physical load can lead to secondary forces induced in members not directly affected by that load; such instances include the effects of relative foundation settlements or differential shortening of adjacent columns connected by stiff elements. However, the physical application of load is not necessary to induce stress in structural elements; changes in temperature (either internal or external) or long-term chemical and physical changes in the material itself can lead to strain in a structural element leading to the potential for induced stress.

The key to self-straining forces is restraint. The differential movement that a member is subjected to, or the internal strain changes within the member itself, will not result in stress unless those strain movements are prevented from freely occurring, and the stiffer the restraint and the larger the movement the larger the stress, and hence force, that will be induced. In some instances, particularly for concrete structures, the induced forces may lead to cracking which, while undesirable from an aesthetic and often a serviceability point of view, may lead to a reduction or elimination of the induced stress altogether. It should be noted that, while it is not discussed in detail here, cracking in concrete members is often the primary consideration when looking at self-straining forces rather than the structural design of members for those forces.

The following sections deal with the different mechanisms of strain and their link to self-straining forces.

10.9.1 Temperature

10.9.1.1 External temperature

When a change in temperature is applied to a physical object, that object will expand or contract linearly to the change in temperature over the normal operating temperature range of structural members, expanding with increases in temperature and contracting with decreasing temperature. This change in length of an element is well known and can be calculated from the following relationship:

$$\Delta L = \alpha \cdot L \cdot \Delta T$$

where ΔL is the change in member length, ΔT is the change in temperature, L the original length and α is the coefficient of thermal expansion for the material under consideration. For concrete, the coefficient can vary considerably depending on the concrete mix and, in particular, the type of aggregate used. A selection of typical ranges for coefficients of thermal expansion is shown in **Table 10.2**.

As can be seen, the coefficients for concrete and steel are similar and any difference is typically ignored when analysing reinforced concrete elements and a single factor is assumed.

As has been discussed already, the two major components of self-straining forces due to temperature are the restraint of the expansion or contraction, and the temperature range an element is subjected to. Common forms of restraint and their impact on structural forces will be discussed in a separate section, but the practical derivation of temperature ranges will be discussed briefly here.

The starting point for the temperature range is the ambient temperature at the time of locking in a structural element to a restraint; for concrete members, this is typically assumed to be the time of casting of the concrete and the ambient 24 hour average temperature is assumed for the time of year the element is cast. The other extreme of the range is then assumed as the maximum and minimum temperature that element is likely to be subjected to. This extreme temperature value depends on a number of factors such as the exposure of the element (whether internal or external) and the possibility of direct solar radiation, the degree of insulation of the member, and the size and thickness of the member itself. Derivation of practical ranges is subjective and relies on statistical analyses to determine likely temperatures for a geographical location according to a specified return period (a similar risk approach is used to determine wind and seismic forces) and a certain amount of 'engineering judgement'. A practical approach, with particular relevance to the UK is presented by the Concrete Society (Alexander *et al*., 2008).

Rules of thumb:

- As a general rule of thumb, the 50 year return temperature ranges shown in **Table 10.3** would be reasonable in the UK for initial design prior to more detailed analysis.
- Appropriate factors of safety would apply to the resultant forces for combination with forces due to other applied loads.

Material	Coefficient (Units: 10^{-6}/°C)
Concrete	6 to 14
Mild steel	11 to 13
Timber	3 to 30

Table 10.2 Typical ranges for coefficients of thermal expansion

Time of installation	Temperature increase (°C)	Temperature decrease (°C)
Winter	+30 (+25*)	0
Spring/autumn	+15	–15
Summer	0	–30

*Temperature increase from winter is taken as +25 due to a minimum temperature of +5°C typically specified as minimum ambient temperature in concrete specifications for casting concrete.

Table 10.3 50 year return temperature ranges

10.9.1.2 Internal temperature

The impact of internal temperature is principally relevant for concrete structures. When a concrete element is cast, the chemical reaction causes relatively high temperatures to develop as the water in the mix reacts with the cement during the hydration process. The maximum temperature reached during the reaction depends upon the materials and their proportions in the mix, the ambient temperature, the size and depth of the pour, and on the presence of any insulation (including formwork). As the temperature begins to cool, and the concrete begins to set, contraction of the concrete will occur in a similar manner as outlined above; if this contraction is restrained, stress will be induced. This is known as early thermal contraction and cracking of the concrete is a typical concern in such situations when contraction of the concrete occurs and is compounded by the relatively low tensile strength of the young concrete.

Rule of thumb:

- The temperature drop from the peak temperature of the concrete during hydration, to the ambient air temperature can be as high as 60ºC and higher depending on the concrete mix, the temperature of the concrete at casting and the thickness of the section being cast. A reasonable initial estimate for temperature drop in a 300 mm deep suspended slab cast on 18 mm plywood formwork and containing 300 kgm^{-3} cementitious material would be of the order 20ºC.

10.9.2 Shrinkage

Shrinkage is a mechanism which affects materials that contain water within their matrix at the time of their installation and which can be lost due to chemical reactions and environmental effects. Concrete, timber and masonry are the three materials principally affected by this mechanism and the overall effect is to reduce the physical size of the member, principally its length, as moisture is lost.

For concrete there are two different mechanisms that fall under the label of shrinkage: autogenous shrinkage, which is particular to concrete, and long-term drying shrinkage which is a similar mechanism in concrete, timber and masonry.

In autogenous shrinkage, water is rapidly drawn from the concrete to participate in the hydration process. This rapid absorption of water from the concrete matrix leads to narrow capillaries forming in the stiffening mix which contract due to surface tension resulting in contraction of the element. This can be prevented by use of chemicals and continuous wet-curing of the concrete surfaces.

Long-term drying shrinkage occurs after the hydration process has finished in concrete and is an ongoing process in concrete, masonry and timber elements, with the total shrinkage dictated by the amount of excess water contained within the element; for timber and masonry, the moisture content at the time of installation is of most interest, and the element may be artificially dried in advance to suit the environment into which it is to be placed (e.g. timber). Water is lost from the material as the unbound water evaporates, with the degree and rate of water loss, and hence shrinkage, related to the relative humidity of the surrounding environment and the size of the element.

Rules of thumb for calculating shrinkage strain can be taken from BS EN1992-1–1:

- For autogenous shrinkage:

  $$\in_{ca} = 2.5 \cdot (f_{ck} - 10)$$

  where f_{ck} is the concrete cylinder strength, and the autogenous strain, $\in_{ca}$ is in microstrain (i.e. 10^{-6}). For 30 MPa cylinder strength, the autogenous strain would be 50 microstrain.
- For long-term drying shrinkage, assuming a concrete with 30 MPa cylinder strength and an average ambient humidity of 60%, the ultimate shrinkage of a 300 mm suspended slab would be of the order 350 microstrain. More than 90% of the strain would be expected to occur in the first three years, with the rate slowing with larger sections.

10.9.3 Structural movement/settlement

10.9.3.1 Foundation settlement

Differential foundation settlement can often, but not always, be avoided. Ensuring that foundation elements are subjected to equal bearing pressures can ensure that differential movement is limited, but it cannot always be avoided entirely. Examples include: construction between new and existing buildings; construction of adjacent elements on different footing types; and settlement of heavy foundation structures, such as beneath tall buildings, causing a dishing of the overall foundation profile following the pressure bulb. Additionally, variable ground conditions and localised pockets of poor soil can lead to differential settlement.

Rules of thumb:

- For low-rise structures, allowable bearing pressures are typically based on a maximum allowable settlement in the region of 25 mm; a reasonable estimate of differential settlement between adjacent footings in this case would be 75% of this total settlement (i.e. just under 20 mm).
- For tall buildings, allowable settlements are typically much greater for practical and financial considerations and are of little concern. In such situations, differential settlements will be necessarily be limited by serviceability considerations and a differential settlement value of the order L/150 could be considered, where L is the distance between adjacent vertical elements.

10.9.3.2 Differential column shortening

In tall building structures, stress differentials in adjacent columns and significant differences in column sizes and shapes can lead to large cumulative differential vertical shortening. Similarly to foundation settlement, the net result is that of a settlement of a support, whether supporting a relatively flexible slab or a stiffer element such as a transfer structure or outrigger.

The shortening of the columns is due to three mechanisms: elastic compression of the column under applied axial load; autogenous and long-term drying shrinkage as previously discussed (in the case of concrete columns); and creep (again, particular to concrete columns). These mechanisms are highly dependent on timing, method and sequence of construction and, in the case of concrete columns, the properties of the mix and the ambient environmental conditions. The third of these mechanisms, creep, will be discussed later.

It is important to note that different calculation strategies are required when assessing shortening of steel and concrete columns. For steel columns, subject only to elastic compression calculations of column shortening need to account for compression of the columns as construction progresses and column elements need to be provided to site making allowance for the pre-compression. For concrete columns, only the movement after construction of a level needs to be considered as slabs are typically cast level (unless a presetting strategy has been adopted to offset shortening); however, the column is subject to shrinkage, creep and elastic shortening.

Calculating differential column shortening can be a complex process, particularly for concrete columns, but the mathematical procedure is well documented (Fintel *et al.*, 1987) and can be modelled through the use of spreadsheets.

Rules of thumb:

- Column shortening is highly dependent upon many factors, and the construction programme in particular has a large impact on final shortening values. For steel columns, the shortening values can be simply calculated from:

  $$\Delta L = \frac{L\sigma}{E}$$

- where ΔL is the change in length, L is the original length, σ is axial stress and E is Young's modulus.
- For concrete columns, stressed at around $0.4 \cdot f_{ck}$, an initial value for total shortening of around 1 mm per metre length of column would be a reasonable initial estimate assuming a high strength

concrete with low water–cement ratio and plasticisers as typically used in tall buildings; for columns under lower axial stress, a proportional value can be taken to derive approximate differentials. Column shortening is typically not assessed in buildings below 70 m (20 storeys) as the differential shortening between columns will typically be relatively low. Maximum shortening will normally occur between two thirds and three quarters the total height of the building.

10.9.4 Restraint

The effects of restraints were discussed briefly earlier; if a movement occurs in a structural element, whether through expansion and contraction due to temperature, shrinkage of the element's material or through differential movement of an element's supports, no stress will be induced in the structural element if that movement is allowed to occur freely, i.e. without restraint. As soon as a restraint is applied, the strain is manifested into a strain and hence force in the element. The degree of restraint and the magnitude of the movement will determine the force induced.

There are several possible causes of restraint:

- internal restraint;
- edge restraint;
- end restraint;
- rotational restraint;
- surface restraint.

Each of these types of restraint will be discussed in relation to the types of movement already defined.

10.9.4.1 Internal restraint

Internal restraint is only typically valid for reinforced concrete. Internal restraint can be caused typically by two sources: reinforcement and temperature gradients. As discussed previously, internal restraint by reinforcement is not a consideration due to temperature effects, as concrete and steel tend to expand and contract similarly under temperature changes. Reinforcement can restrain the concrete when the concrete section is subjected to shrinkage, however; as the concrete contracts, the internal bond friction generates a stress transfer between the two materials. As the concrete contracts, the reinforcement is put into compression and, if there is no net compression in the element, the concrete goes into tension. If reinforcement is placed asymmetrically in a section, the uneven restraint can also induce a deflection into the member.

In large concrete sections, as the concrete cools down from the peak temperatures of the hydration process, the surface temperature cools much more rapidly than the core. As a result, the surface areas of concrete contract much more rapidly than the core leading to the different layers of concrete restraining one another; this can lead to cracking of the surface concrete. Careful specification of the concrete mix and a robust method statement need to be adopted to ensure that any cracking is limited and controlled

10.9.4.2 Edge restraint

Edge restraint is again typically valid for concrete elements. When a section of wall or slab is cast integrally against adjacent concrete, the differential shrinkage of the two elements due to their different ages and hence different stages of shrinkage, leads to restraint of the 'newer' concrete by the 'older' concrete. The situation is most critical when new concrete is cast integrally against an existing structure. Typically, the main concern with edge restraint is the potential to cause cracking of the concrete. With careful construction sequencing and design of reinforcement to deal with any remaining restraint issues, the level of cracking can be controlled.

10.9.4.3 End restraint

End restraint of structural elements can lead to significant self-straining forces; if opposing stiff elements prevent the element from contracting or expanding freely, those elements will be subjected to an imposed movement leading to forces being induced. Opposing stiff elements can consist of opposing core walls and stiff columns, etc. The restraint to movement will also lead to a compression or tension induced within the restrained element that will need to be considered in the design. For steel elements, compression due to restraint of expansion could lead to buckling of the member, and in concrete, restraint of contraction could lead to cracking.

Rule of thumb:

- Movement joints at 50 m centres will alleviate the potential for the majority of self-straining forces. Where very stiff elements directly oppose one another and a movement joint is not feasible, a simple model of the restraints and the restrained element can be built, accounting for any cracking, and a contraction and/or expansion applied in the analysis based on the rule of thumb data supplied for each of the self-restraining actions above; the resultant element forces can then be used for design.

10.9.4.4 Rotational restraint/rigid link

Where vertical elements are linked together horizontally by stiff elements, such as continuous transfer structures or outriggers rigidly connected to a lateral stability core, differential movement due to foundation settlement or column shortening as discussed above can lead to large induced forces in the horizontal elements due to the 'imposed' displacement.

Using the unrestrained differential settlement as the imposed displacement, such as using the rule of thumb figures defined in the sections above, is typically very conservative. This is due to two reasons: firstly, as the link element is displaced, shear is transferred along the member from the end that settles most leading to a redistribution of load in the vertical elements and a reduction in the differential; secondly, for concrete elements, creep of the link member occurs under long-term loading and the load transfer is reduced.

Rule of thumb:

- For both concrete and steel link elements, a reasonable rule of thumb is to assume an imposed displacement equating to 50% of the unrestrained differential settlement when analysing the link element.

10.9.4.5 Surface restraint

Again, in common with many types of restraint, surface restraint is particularly relevant for concrete elements. Where concrete is cast upon a different material, such as onto the ground, as a topping to a precast element, or onto a composite steel deck, the differential contraction of the concrete due to shrinkage, temperature or a combination of the two may lead to tension forming in the concrete element as it is restrained by the surface it is laid upon. Cracking is the typical outcome where this is not guarded against. Where concrete is cast upon a permanent substratum, such as directly on the ground, the use of a double layer of high density polythene will typically be sufficient to act as a slip membrane and negate the restraint.

10.9.5 Parasitic (secondary) 'loads' due to pre-stressing

In a typical single span pre-stressed element (say a bridge beam), when the pre-stressing forces are applied the element will displace freely upwards and the internal force distribution is directly determinable from the internal actions of the pre-stressing plus the externally applied forces.

For elements which are part of statically indeterminate structures the movements of the elements are constrained by the restraints, and hence parasitic or secondary (which has an unfortunate connotation of being secondary in magnitude so not preferred) forces develop. The forces which occur at the supports modify the reactions under external loads and moments are developed in the structure as a consequence.

It is important to understand that the parasitic forces developed are relevant only at the serviceability limit state, as they represent variations in the internal distribution of stresses; the ULS forces and moments are directly determined simply through the application of the external loads to the structural model.

For pre-stressed concrete design in particular explicit allowance needs to be made for the stresses at SLS, as opposed to conventional reinforced concrete (RC) where the stress state at SLS is assumed to be acceptable so long as the element's ULS strength is sufficient.

10.10 Fire loads

10.10.1 Fire loads and fire limit state

Fire effects on structures have been traditionally dealt with using methods which assumed a certain 'deemed to satisfy' resistance to a 'standard fire'. Recently more sophisticated approaches have been developed, which seek to provide a more flexible (but more complex) set of methods to design structures for fire effects.

It can be confusing to talk of 'fire loads' for structures, since it is a term used to denote the energy which is contained in the combustible materials in a particular fire compartment, which can be expressed as a 'wood equivalent' but is more commonly given in MJ/m^2. The fire load design values are base on surveys and are normally given for different statistical fractile values of which the 80% fractile is commonly used for design. These design fire load density values can range between 350 MJ/m^2 for a school classroom and up to 1900 MJ/m^2 for a traditional library. However, it is also possible to calculate the actual fire load in a compartment based on the calorific values of the different materials present. Based on the 'fire load', the available ventilation and the insulating properties of the compartment envelop, different calculation approaches can be used to calculate the gas temperatures in an individual fire compartment. Once the temperature in the environment is known it is possible to calculate the temperature of the structural members affected by the fire and then a reduced capacity of the members for the temperature can be calculated.

The capacity of the members and connections in a structure are compared against a reduced set of load effects, reduced in order to take into account the 'accidental' nature of the fire load effect. This design scenario is call 'fire limit state' and essentially means that the permanent (dead) load of the structure must be resisted, along with a reduced live load (reduced depending upon the probability of the specified live load being present) – higher for say a storage load compared to say a roof, as well as a portion of snow load where applicable and wind load. It is also important that the impact of the restraint of the structure (either internal restraint or external restraints) due to the expansion of the structure under the change in temperature due to the fire be taken into account (in a similar manner to the effects of shrinkage and temperature in general). Furthermore, in some instance it may be required to consider the non-uniform heating of structural members and the resulting internal stresses and deformation of the members (particularly important for material with a low conductivity, for localised fire and for slender compression elements).

To calculate the response of a structure to fire a number of approaches of different complexity are available ranging from empirical table values, over simple analytical equations based on high temperature material properties to full finite element programs able to simulate the real behaviour of large parts of structure based on first principle mechanics. A good set of material properties, design approaches and equations is provided in the 'hot' parts (EN199x-1–2) of the structural Eurocodes for all commonly used construction materials.

Further details and information on fire loads and the prediction of the structural response at high temperatures can be found in the two IStructE guides to fire safety of structures.

10.11 Fluid loads

Generally we classify materials as either solids or fluids. Fluids are materials that flow, and are therefore either liquids or gases. Typically when engineers consider fluid loads acting on structures we consider only the static pressures which are applied.

The static pressure of a retaining structure of a liquid 'at rest' is simply equal to the density of the fluid times the distance to a free surface. This gives us the classic 'triangular' lateral distribution of pressure. The most commonly encountered case of this is water pressure, either on a basement wall, or on the walls of a water tank.

Fluid dynamics is more complex and is typically limited to marine or civil engineering structures, such as culverts, bridge abutments, weirs, docks, etc. Essentially if there is a 'flow' of water and that flow is either confined or expands there will be pressures developed laterally. The magnitude of the forces depends upon the speed of the flow and the degree of confinement or expansion of the flow.

In addition to these dynamic fluid loads there is also the impact of kinematic or impact fluid loads – for example a wave breaking on a 'sea wall', where the kinematic pressure may be 10 times that of the hydrostatic pressures. The height of the water level, height of waves, and geometry of the sea wall affect the loads.

Seismic design of retained fluids is also complex, with specialised codes being used. Typically the fluid is converted into an equivalent static mass, with a smaller dynamic mass above – representing the 'sloshing' part of the fluid. Reference should be made to specialist literature for the design of tanks subject to seismic loads.

10.12 Silo loads

Silos are typically tall cylindrical structures used for storing granular materials. The behaviour of relatively deep structures retaining materials differ from the 'fluid-like' assumed behaviour discussed above.

In silos with material 'at rest' there is an activation of frictional resistance between the granular material and the wall; this produces vertical load in the walls but a counter-balanced reduction in the lateral loads. The lateral pressure on the walls varies according to the density of the retained material, the hydraulic radius, the friction coefficient between the material and the wall and the angle of internal friction of the retained material. The key difference between fluid and silo loading is that the pressures exerted are not linearly variable with depth due to the arching action of the retained material.

For silos which are being unloaded the distribution of loads varies depending upon both the manner of the unloading (i.e. from top, or from the centre of the bottom). If unloaded from the top the lateral pressure remains similar to that during loading (see above), but if unloaded from the centre of the bottom there are two alternative behaviours: 'core flow' or 'mass flow'. In 'core flow' there is a central core which extends from the top surface to the opening and the material effectively follows that core – there is an increase in lateral pressures from an 'at rest' filled position. With 'mass flow' typically this is with silos with steep sides which are 10–15 degrees from the vertical and are designed to have the entire mass moving together. This sort of flow causes high local lateral pressures at the point of changing from vertical sides to inclined sides due to the change from a uniform vertical flow to an accelerating confined flow. It should be noted that 'mass flow' can occur in any type of silo due to formation of effectively an internal 'virtual' hopper shape inclined 10–15 degrees to the vertical (with the remaining mass remaining effectively static). The rule of thumb is that if the height retained material in a silo is more than four times the hydraulic radius then 'mass flow' is possible.

For some types of retained materials and silos (such as cement silos) it is possible for the materials to become 'fluidised', effectively acting as fluids and applying lateral pressures accordingly. This tends to occur when the filling of the silo occurs quickly, or where the retained materials are aerated to ease unloading.

Dealing with asymmetric loading or unloading is more complicated and should generally be avoided. Key points are:

- Lateral loads are developed due to the friction between the walls and the retained material.
- Using realistic values for the density of the retained materials is important – for instance the values of densities given for grains may be exceeded significantly due to the presence of water; also if there is vibration of the silo the retained materials may become densified.
- The angle of repose, the angle of internal friction and the angle of wall friction are important parameters; the friction between the material and the wall will be affected by the material and over time with wear.
- Various methods and devices are available which increase the speed or loading or unloading silos; it is important to know whether any of these are to be used.
- For an economical design it is important to understand the operational use of the silo to be designed; designing based upon a combination of 'worst case' design parameters may be required if there is no confidence in the ability to control the operation of the silo.
- Silo loading, particularly if vibration (in service or due to seismic action) needs to be considered, is complex and reference to specialist literature is recommended when designing a silo.
- It is important to understand the operational cycle of loading and unloading of the silo in practice, since the design loading pattern needs to consider the worst case to be experienced in service.
- Unless the retained material properties are well understood material testing may be required.
- Consideration of an 'envelope' of potential material properties is regarded as 'good practice'.
- Explosion and design of relieving measures should always be considered and allowed for in the design.

- When seismic effects have to be considered it is important to understand that the seismic effects are more akin to an inverted triangle of forces, rather than a uniform load (i.e. not equivalent to wind loading – there is a higher relative moment generated under seismic loads than would be generated by taking the base shear and assuming it acts at the centre of mass).
- There is a need to specify and control the (internal) finishes for assumptions related to friction to be valid.
- Differential settlement of supports should be considered.
- Various codes can be used and these typically include geometrical limits on silos and their openings (to ensure that the code design approaches are 'valid').

10.13 Soil/earth loads

Generally the structures designed by structural engineers which are loaded by soil or earth fall into the following categories:

- cantilever retaining walls;
- propped retaining walls (basement walls).

For the calculation of loads it is generally the active pressures which are considered. For liquids, this loading is as per fluid loads above and is simply a combination of the density of the liquid and the depth from a zero pressure level as the pressure. For granular soils the pressure is generally taken as being a fraction of equivalent liquid pressure, with a coefficient related to the internal angle of friction (angle of repose) and the soil/wall friction. Typically the vertical friction 'load' on the wall is neglected. The active pressure developed for a cohesive soil differs. Generally unless specific guidance is provided related to the soil properties simple assumptions are used (see rules of thumb below); this is particularly the case where the designer is not closely involved in the construction.

Active pressures are the 'loads' applied by a retained or contained soil. The resistance of the soil mass to counterbalancing loads (i.e. at the toe of an embedded retaining wall where the wall is 'pushed' against the soil by the retained mass) is the passive resistance. Passive pressures for granular and cohesive soils are generally higher than active pressures and need to be considered carefully before being used (i.e. they are potentially non-conservative, particularly if the soil mass may be removed or if for example with cohesive soils they become very dry and a 'gap' develops between the wall and the soil, requiring a large movement of the wall before the passive resistance can develop).

Typical formulas are given below.

10.13.1 Static lateral loads from soil

The uniform surcharge load is assumed to be 10 kN/m^2.

For walls laterally supported at top and bottom, and assuming lateral strain in the soil is zero, the at-rest earth pressure at depth z is given by:

$p_o = 0.5(20z + 10)$ kN/m^2 above the water table, and

$p_o = 0.5(20z - 10(z\text{-}2) + 10)$ kN/m^2 below the water table.

For cantilever retaining walls, the active earth pressure is given by:

$p_a = 0.33(20z + 10)$ kN/m^2 above the water table, and

$p_a = 0.33(20z - 10(z\text{-}2) + 10)$ kN/m^2 below the water table.

10.13.2 Allowable passive resistance provided by soil

Groundwater level in front of a retaining structure will depend upon the degree of drawdown produced by local dewatering array. Assume groundwater level to be at d_w mBGL with no seepage taking place. Excavation in front of the wall is assumed to be up to 1 m depth.

For cantilever retaining walls, the passive resistance at depth z is given by:

$p_p = 3[20(z - 1)]$ kN/m^2 above the groundwater level, and

$p_p = 3[20(z - 1) - 10(z - d_w)]$ kN/m^2 below the groundwater level.

Rules of thumb:

- Where considering soil as a dead load generally a value of 20 kN/m^2 is suitable.
- For design of retaining walls the minimum active pressure can be found by treating the soil as liquid with a density of 5 kN/m^2; for retaining walls not higher than 5 m this will normally be the controlling load case. For gravel soils Ka = 0.27 and if the average bulk density = 20 kN/m^3; Pa = 0.27 × 20 = 5.4 kN/m^3. Therefore it is conservative to use Ka = 0.27 for both clay and gravel soils.
- A hydrostatic water pressure acting over 1/3 the height of the wall should always be applied even if the wall is drained and there is no groundwater shown by the site investigation. For soils with high water tables a full hydrostatic head should be assumed.
- For propped basement walls <7 m high where backfill has been compacted behind the wall, a uniform pressure of 35 kN/m^2 should be applied to the full height of the wall.
- For propped retaining wall construction, i.e. a basement wall, earth pressure at rest Ko should be taken as 0.7.
- The usual assumption made in the absence of a detailed knowledge of the actual groundwater conditions is that the water table is located 1 m below the ground surface.
- If the water table level is near the ground surface it is acceptable to consider the factored (Ultimate Limit Strength) hydrostatic pressure as being that which considers water at ground level, or otherwise the 'maximum credible' value of the water pressure.
- Some idea of the variation to loading based on different soils can been reached by considering – Phi of 30 degrees for 'normally consolidated loose sands', 35 degrees for medium dense well graded sand, typical clay Phi = 25 degrees.
- For t-section retaining walls, free to move at the base – design the wall stem using Ko, for overturning and sliding use Ka.
- For embedded cantilever walls use Ka.

- For propped embedded walls use Ko.
- For overconsolidated clays (such as found in the UK) typical Ko is 1.5.
- For overconsolidated sands Ko around 1.0.
- Water levels are far more critical than the actual Phi of a soil (hence use 30 degrees for 'first concept' work).

10.14 Conclusions

Dr E. H. Brown defines structural engineering as 'The art of moulding materials we do not really understand into shapes we cannot really analyze, so as to withstand forces we cannot really assess, in such a way that the public does not really suspect.' Although he wrote this in 1967 and significant progress has been made in terms of understanding materials and forces and developing increasingly complex methods for analysis of structural behaviour we must recognise that as professional engineers we must be constantly aware of the limits in our understanding of 'real' structural behaviour. The art of engineering needs to remain a subject of engineering judgement and not adherence to limited and limiting codes of practice. The determination of loads to be used in any structural design should be based upon the best possible understanding of the past, present and future of the building being considered and reasoned and consistent judgement.

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10.15.1 Useful websites

www.steelconstruct.com

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CONTENTS

Chapter 11

Structural fire engineering design

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The purpose of this chapter is to explain the methodology underpinning the structural fire engineering design process. Structural fire engineering design consists of three basic components: choosing an appropriate design fire, using this information to derive the temperatures of the structural elements and assessing the structural behaviour with respect to the temperatures derived. For each element of the structural fire engineering design process there are a number of options available to the designer depending on the complexity of the project, the state of knowledge with regard to the structural material chosen and the objectives of the fire engineering design strategy. Detailed information on the design methodology in this area is available in the Institution of Structural Engineers' *Guide to the Advanced Fire Safety Engineering of Structures* (2007).

11.1 Introduction

The traditional means of ensuring compliance with the requirements of the Building Regulations for structural fire safety is to rely on the results from standard fire tests on individual elements or components. At the simplest level structural fire engineering is based on simple prescriptive rules and guidance which ensure sufficient passive fire protection is applied to structural members or that minimum dimensions are satisfied to ensure load-bearing capacity and/or the separating function is maintained for a period corresponding to the recommended fire resistance requirement from the regulatory guidance.

In this way, structural engineers have been involved in fire engineering for many years without necessarily being aware of it and most probably being unaware of the background to the development of the regulations and the guidance that underpins them. For example, a structural engineer responsible for designing a reinforced concrete framed building will specify the overall dimensions, size and position of reinforcement dependent on the ambient temperature design considerations in terms of loading and environmental conditions. In the vast majority of cases, the structural fire engineering will simply consist of checking in the tables produced in BS 8110 Part 2 to ensure that the design meets the minimum dimensions and minimum depth of cover to the reinforcement for the specified fire resistance period.

Within this simple process there are a large number of implicit considerations on the likelihood of a fire occurring: the consequences in terms of life safety should a fully developed fire occur, the thermal exposure within the fire compartment and the consequent temperature distribution through the structural member. To a large extent structural fire engineering design simply consists of making explicit decisions rather than relying on the implicit assumptions within the prescriptive approach.

The three-stage approach to structural fire engineering design is illustrated schematically in **Figure 11.1**.

11.2 Compartment time–temperature response

The first step in a structural fire engineering design is to evaluate an appropriate compartment time–temperature response to be used for the subsequent heat transfer and structural response calculations. This initial process can itself be further subdivided into two important preliminary tasks: the choice of appropriate design fire scenario(s) and the selection based on the design fire scenarios adopted of an appropriate design fire.

11.2.1 Design fire scenario(s)

The appropriate design fire scenarios should be determined on the basis of an overall fire risk assessment taking into account the nature and distribution of fire load within the project and the

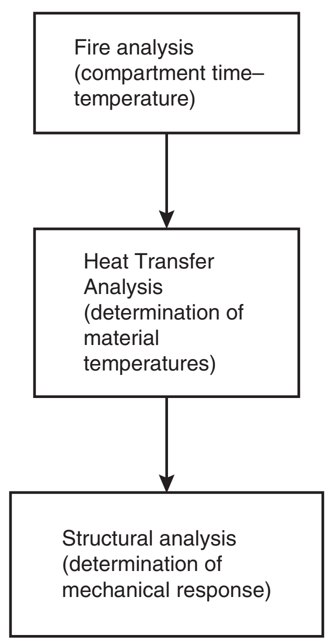

Figure 11.1 Three stages of structural fire engineering design

presence of likely ignition sources and the impact of detection and suppression systems.

The design fire scenarios selected will identify specific compartment geometries with their own associated fire loads and ventilation conditions and should be based on a 'reasonable worst case scenario'. The choice of design fire scenario will dictate the choice of the design fire to be used in subsequent analysis.

To take a simple example, an appropriate design fire scenario within a medium rise residential building consisting of a number of separate dwellings would be a fire within a single dwelling bounded by fire resisting construction. Given the presence of sufficient oxygen for combustion, sufficient fire load and an ignition source a fully developed fire within a single dwelling would be one design fire scenario to be considered.

11.2.2 Design fire

For each design fire scenario adopted, a design fire will be chosen that represents the likely risk within that area. Normally the design fire is only applied to one fire compartment at a time, i.e. in the example above it would not be normal practice to assume that two dwellings were fully involved in a fire at the same time. The extent of the fire to be considered will, to a large extent, be governed by the compartmentation in place within the building.

This stage of the process involves the selection of an appropriate *model* representing the fire within the compartment under consideration. In many cases, the type of occupancy will play a major role in defining the type of model to be used. Given a fire load and an ignition source there are three options in terms of fire development: either (i) the fire is extinguished due to manual or automatic suppression or lack of oxygen, (ii) the fire remains localised due to a lack of oxygen or insufficient fuel load or (iii) the fire becomes fully developed. For the designer, detection and the active intervention of third parties (such as the Fire and Rescue Service) are not taken into account, therefore the chief consideration is to decide if the fire will remain localised or grow into a fully developed fire. In terms of structural considerations, the most serious situation is where flashover occurs within the compartment and all combustible materials become involved in the process. Such a situation would require the adoption of a post-flashover fire model.

Combustion behaviour within a fire compartment is a complex process involving a mass balance where the energy released from combustion of the fire load is utilised in convective heat flow through openings where hot gases inside the compartment are replaced by incoming cold air, radiated heat flow through the openings and heat losses to the compartment boundaries. For uncontrolled compartment fires this complex process can be simplified into a three phase behaviour characterised by the transition point known as flashover. Compartment fire behaviour is illustrated schematically in **Figure 11.2**.

Localised fire models are available in codes and standards but are not considered further here as, for structural fire engineering it is the post-flashover situation which represents the most serious threat to structural stability.

The principal choice facing the designer at this stage of the process is whether to use a nominal fire curve or a 'natural' fire model to evaluate the compartment time–temperature response. Nominal fires are representative fire curves for the purposes of classification and comparison but bear no relationship to the particular characteristics of the building under consideration. Natural fires are calculation techniques based on a consideration of the physical parameters specific to a particular building or fire compartment. **Figure 11.3** illustrates the options available to the designer when choosing to model compartment time-temperature behaviour.

11.2.2.1 Nominal fire curves

Nominal or standard fire curves are the simplest and most commonly adopted means of representing a fire. They have been developed to allow classification and assessment of construction products using commercial furnaces. Although they do not represent 'real' fire scenarios they have been developed from experience of real fires. A number of different curves exist. The choice of curve for a particular situation will depend on the

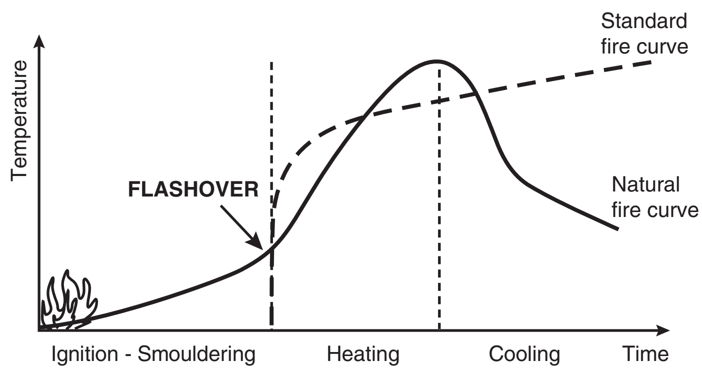

Figure 11.2 Three phase fire behaviour

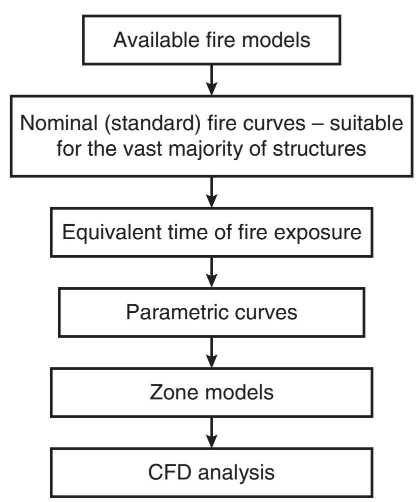

Figure 11.3 Available options for modelling fire behaviour in order of increasing complexity

end-use. Different curves are used for testing and assessment depending on whether the structural element or product is to be used in the construction of a normal building (office, dwelling, etc.), the petrochemical or offshore industry or for tunnels.

The most well known and widely adopted nominal fire curve is the so-called ‘standard’ fire enshrined in National, European and international standards. The standard fire curve is based on a cellulosic (i.e. wood/paper/fabric) fire within a compartment and is described by the following equation:

$$\theta_g = 20 + 345 \log_{10} (8t + 1) \qquad (11.1)$$

As with many other nominal fire curves it is characterised by a steadily increasing temperature and does not incorporate a descending branch or cooling phase.

The standard curve has been adopted throughout the world for a number of reasons: to provide evidence of regulatory compliance; to assist in product development; and to provide a common basis for research into the effect of variables other than temperature. As such it has proved remarkably successful over a long period of time. It has the advantage of familiarity for designers, regulators and specifiers. The existence of a large body of test data facilitates the continuing use of the standard curve and enables tabulated data for generic materials to be developed. It is simple to use and clearly defined and allows for a direct comparison of the performance of products tested under nominally identical conditions.

However, the standard fire test suffers from a number of drawbacks when any attempt is made to extrapolate test results to performance in real life situations. These drawbacks arise as a consequence of simplistic assumptions regarding the thermal exposure and the support and loading conditions of the test specimen. Whilst the standard curve incorporates the transient nature of fire development there is no direct relationship between performance in a standard test and the duration of a real fire. This is a source of some confusion as many observers conclude that 60 minutes’ fire resistance means that the element of structure will survive for 60 minutes in a real fire. In reality, the element of construction may perform satisfactorily for a longer or shorter period depending on the severity and duration of the fire and the boundary conditions and loading present in the building at the time of the fire. The temperature within a furnace is relatively uniform compared to the temperature within a real fire compartment. Spatial temperature differences (particularly during the growth face) may lead to longitudinal and cross-sectional thermal gradients within structural members that are not present during a furnace test which in turn could lead to deformations not observed during a standard test. For certain forms of construction, direct flame impingement during a real fire may have important implications which cannot be observed in a standard test. As mentioned above, a real fire consists of three distinct phases. The relative durations of these three phases may have a significant impact on the performance of elements of structure. Such behaviour cannot be addressed by an ever increasing curve where temperature rises at a decreasing rate with time.

In addition to the problems associated with the relationship between the standard thermal exposure and real fires, a number of difficulties arise in extrapolating the results from standard tests to predict structural behaviour under realistic conditions. The geometric limitations of specimen size mean that it is not possible to simulate complicated three-dimensional structural behaviour. No allowance can be made during the test for the beneficial or detrimental effects of restraint to thermal expansion provided by the surrounding cold structure. The nature of the test means that only idealised end conditions can be used and only idealised load levels and distributions are adopted. During a fire some degree of load shedding will take place from the areas affected by fire to the unheated parts of the building. In the standard test, no allowance can be made for alternative load-carrying mechanisms or alternative modes of failure that are a function of the building rather than the element of structure. In particular, the standard fire test does not address the important role that connections play in maintaining overall global structural stability.

A reliance on the results from standard tests and, in particular, the use of tabulated values for generic products has retarded our understanding of structural behaviour in fires. Structural fire engineering attempts to go beyond a blind reliance on prescriptive guidance, to consider the physical characteristics that contribute to fire development and evaluate the material and mechanical response of the structure to the increase in temperature.

Although the ‘standard’ fire curve is the most well known a number of other nominal curves exist for special circumstances. These include the external fire curve where the structural element is subject to heating from flames emerging from openings. For situations such as petrochemical plants where the calorific value of combustibles is significantly higher than the cellulosic material assumed for normal building design a number of hydrocarbon fire curves exist. In recent years a number of high profile tunnel fires have caused great damage and loss of life. In such applications an even more severe exposure than the hydrocarbon curve may be appropriate to simulate the effect of a fire involving large petrol tankers in a confined space. The most onerous exposure has been developed in the Netherlands as the RWS curve which reaches temperatures of 1350ºC. Other curves include the German RABT curve which achieves a maximum temperature of 1200ºC.

11.2.2.2 Natural fire models

All of the nominal fire curves discussed above are post-flashover models of fire behaviour under various conditions. They are models loosely based on observed behaviour in real fires but are not based on any physical parameters. Natural fire models are based on the physical parameters that influence fire growth and development and range from simple models for both localised fires and post-flashover fire behaviour to advanced methods based on computational fluid dynamics. The remainder of this section deals with simple post-flashover calculation models for establishing compartment time–temperature response.

A number of attempts have been made to utilise the simplicity of the standard fire curve and to relate actual fire severity to an equivalent period within a standard test. Time equivalence is an extremely useful tool for demonstrating compliance with regulations in a language clearly understood by building control authorities. The basic concept considers equivalent fire severity in terms of the temperature attained by a structural element within a fire compartment and the time taken to achieve the same temperature in a standard fire test. The concept is illustrated in **Figure 11.4**. Alternative formulations consider the normalised heat input from a standard furnace. The vast majority of the research effort into time equivalence has been initiated by the steel industry and the results are therefore largely applicable to protected steel specimens. However, if the data exist, there is no reason why the concept should not be extended to cover other forms of construction.

The concept of time equivalence relates the severity of a real compartment fire in an actual building to an equivalent period of heating in a standard furnace test. This equivalent period is then compared with the design value of the standard fire resistance of the individual structural members, which must satisfy the following relationship:

$$t_{e,d} < t_{fi,d} \tag{11.2}$$

where, $t_{e,d}$ is the design value of time equivalence and $t_{fi,d}$ is the design value of the fire resistance of the member. A number of methods are available to calculate time equivalence. The most

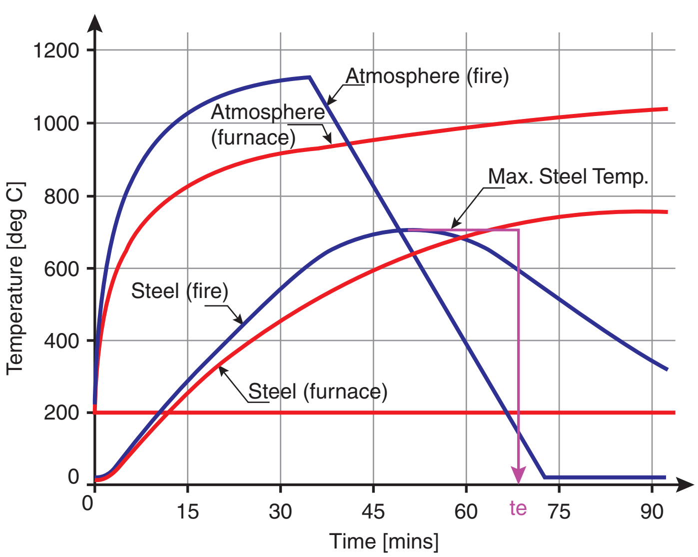

Figure 11.4 Graphical representation of the concept of time equivalence (t_e)

widely used is that in the fire part of Eurocode 1 which is of the form:

$$t_{e,d} = (q_{f,d} \times w_f \times k_b) \times k_c \tag{11.3}$$

where:

- $q_{f,d}$ is the design fire load density per unit floor area (MJ/m²)
- k_b is the conversion factor for the compartment thermal properties (min.m²/MJ)
- w_f is the ventilation factor
- k_c is a correction factor dependent on the structural material

Detailed guidance is available on the use of the method (Lennon *et al*., 2006).

The time equivalent method represents a sort of 'halfway house' between nominal and natural fire models to describe severity in a language understood by designers, manufacturers and regulators. A more rational approach is to consider fire behaviour purely in relation to the factors that influence fire growth and development independent of any reference to standard test procedures. A number of simplified models exist to calculate the time–temperature response caused by a fire within a building compartment. The most commonly used and widely validated method is the parametric approach set out in the fire part of the Eurocode 1 for Actions on Structures. The temperature-time curves in the heating phase are given by:

$$\theta_g = 1325(1 - 0.324e^{-0.2t^*} - 0.204e^{-1.7t^*} - 0.472e^{-19t^*}) \tag{11.4}$$

where:

θ_g = temperature in the fire compartment	(ºC)
t^* = t.Γ	(h)
t = time	(h)
$\Gamma = [O/b]^2/(0.04/1160)^2$	(-)
$b = \sqrt{\rho c \lambda}$ and should lie between 100 and 2200	(J/m²s^{½}K)
O = opening factor $(A_v \sqrt{h} / A_t)$	(m^{½})
A_v = area of ventilation openings	(m²)
h = height of ventilation openings	(m)
A_t = total area of enclosure (including openings)	(m²)
ρ = density of boundary enclosure	(kg/m³)
c = specific heat of boundary enclosure	(J/kgK)
λ = thermal conductivity of boundary	(W/mK)

The theory assumes that temperature rise is independent of fire load. In order to account for the depletion of the fuel or for the active intervention of the Fire and Rescue Service or suppression systems, the duration of the fire must be considered. This is a complex process and depends on the rate of burning of the material which itself is dependent on the ventilation and the physical characteristics and distribution of the fuel.

The parametric approach is a relatively straightforward calculation ideally suited for modern spreadsheets. It provides a reasonable estimate of the average time–temperature response for a wide range of compartments and represents a major advance compared to a traditional reliance on nominal fires which bear little or no relationship to a realistic fire scenario. The parametric fire curves comprise a heating phase represented by an exponential curve up to a maximum temperature θ_{max} occurring at a corresponding time of t_{max}, followed by a linearly decreasing cooling phase.

The maximum temperature in the heating phase occurs at a time given by:

$$t_{max} = \max[(0.2 \times 10^{-3} \times q_{t,d}/O_{lim}); t_{lim}] \tag{11.5}$$

where:

$$q_{t,d} = q_{f,d} \times A_f / A_t$$

and t_{lim} = 25 min for a slow fire growth rate, 20 min for a medium fire growth rate and 15 min for a fast fire growth rate.

For most practical combinations of fire load, compartment geometry and opening factor t_{max} will be in excess of these limiting values. The temperature–time curves for the cooling phase are then given by:

$$\theta_g = \theta_{max} - 625(t^* - t^*_{max}) \text{ for } t^*_{max} \le 0.5(h) \tag{11.6}$$

$$\theta_g = \theta_{max} - 250(3 - t^*_{max})(t^* - t^*_{max}) \text{ for } 0.5 < t^*_{max} < 2(h) \tag{11.7}$$

$$\theta_g = \theta_{max} - 250(t^* - t^*_{max}) \text{ for } t^*_{max} \ge 2(h) \tag{11.8}$$

The relevant input parameters for the parametric approach are illustrated schematically in **Figure 11.5**.

The concept of time equivalence and parametric fire exposure is illustrated by reference to a simple worked example below.

Time Equivalence Design information:

Compartment in 4 storey office building

Floor area: A_f = 6 m × 6 m = 36 m²

Design fire load density: = 570 MJ/m² (80% fractile value for offices from PD 6688-1-2: 2007)

Compartment construction: roof formed from hollowcore concrete slabs, walls and floor lined with plasterboard

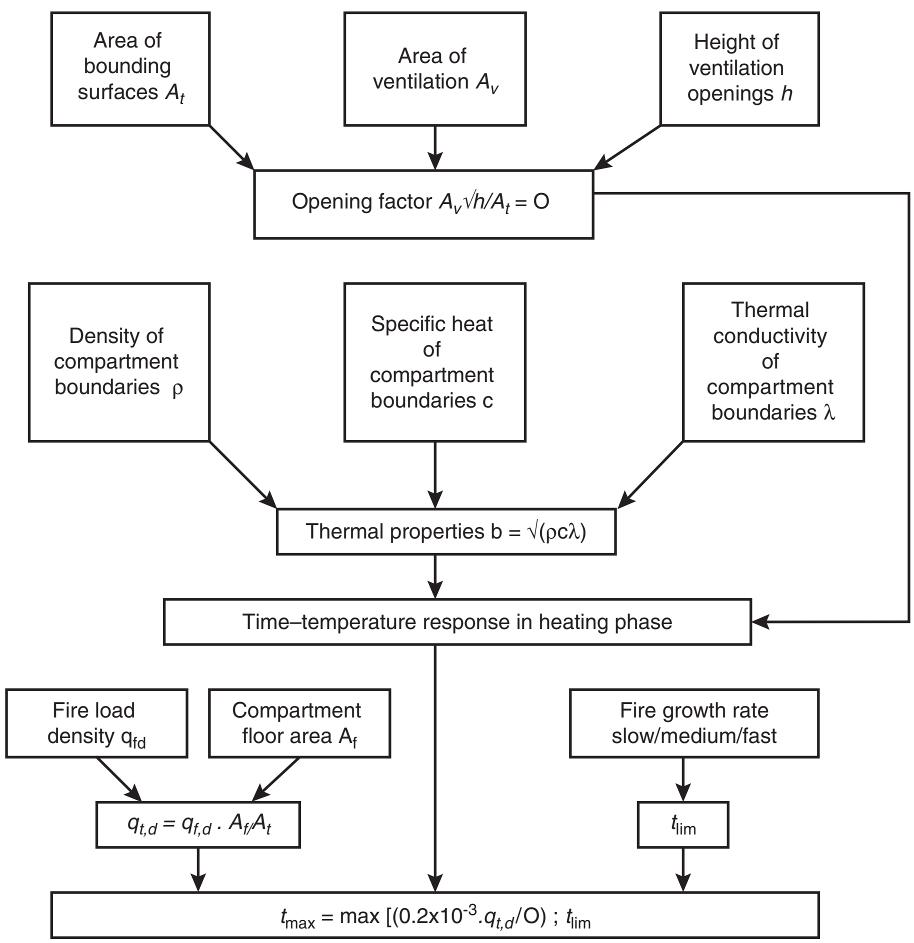

Figure 11.5 Input values for parametric calculation

Ventilation area $A_v = 3.6 \text{ m} \times 2 \text{ m} = 7.2 \text{ m}^2$
Height of compartment H (m) = 3.4 m
Total area of enclosure $A_t = (2 \times 6 \times 6) + (4 \times 3.4 \times 6) = 153.6 \text{ m}^2$
Opening factor $O = A_v \sqrt{h} / A_t = 7.2 \times \sqrt{2} / 153.6 = 0.066 \text{ m}^{-1}$
Calculation:
Ventilation factor: $w_f = (6/H)^{0.3}[0.62 + 90(0.4 - \alpha_v)^4] \geq 0.5$
$\alpha_v = A_v / A_f = 7.2/36 = 0.2$ (this is within the limits in the Eurocode)
giving $w_f = 1.95$
Thermal properties of compartment linings: The factor k_b is dependent on the thermal inertia of the construction materials as defined by the factor $b = \sqrt{\rho c \lambda}$ where:
ρ = density (kg/m³)
c = specific heat (J/kgK)
λ = thermal conductivity (W/mK)
Although no information on the thermal properties of commonly used construction materials is provided in the Eurocode (or the National Annex and associated NCCI), some guidance is available in the literature. **Table 11.1** sets out the appropriate values for the current case taken from published data.

The b value to be used for design is a weighted average where $b = \sum b_j A_j / A_j$. Here the relevant b value = 945J/m²s^{½}K. From Table B.1 of the NCCI this corresponds to a value of $k_b = 0.07$. Note: If no detailed information is available on the thermal properties of the compartment linings or if there are uncertainties about the final construction or changes may be made over the course of the building's design life then the default value of $k_b = 0.09$ should be used.

The equivalent time of fire exposure is then given by:

$$t_{e,d} = 570 \times 1.95 \times 0.07 = 78 \text{ min} \qquad (11.9)$$

The above example of a corner office compartment is used to illustrate the parametric approach.

Design information:

Floor area $A_f = 36$ m²

Design fire load density = $q_{f,d} = 570$ MJ/m²

Opening factor O = 0.066 m^{-1}

Thermal inertia $b = 945$ J/m²s^{½}K

Construction	Material	Thermal inertia (b value – J/m²s^{½}K with $b = \sqrt{\rho c \lambda}$)	Area (m²)
Ceiling	Concrete	2280	36
Floor	Plasterboard	520	36
Walls	Plasterboard	520	76.8

Table 11.1 Thermal properties of compartment linings

The parametric time factor Γ is a function of the opening factor O and the thermal inertia b

$$\Gamma = (O/b)^2/(0.04/1160)^2 = (0.066/945)^2/(0.04/1160)^2 = 4.1$$

Fire load

$q_{f,d} = 570$ MJ/m²

$q_{t,d} = q_{f,d} \times A_f/A_t = 570 \times 36/153.6 = 133.6$ MJ/m²

Maximum temperature will be at time

$$t_{\max} = (0.2 \times 10^{-3}\, q_{t,d} / O) = 0.2 \times 10^{-3} \times 133.6/0.066 = 0.4 \text{ hours (24min)}$$

The heating and cooling phases can then be constructed using the relevant formulae above to give the compartment time-temperature response illustrated in **Figure 11.6**.

11.3 Heat transfer

Heat transfer analysis is undertaken to determine the temperature rise and distribution of temperature within the structural members. Thermal models are based on the acknowledged principles and assumptions of heat transfer. They vary in complexity ranging from simple tabulated values to complex calculation models based on finite difference or computational fluid dynamics. The heating conditions considered extend to cover natural fire scenarios. However, the validity of some of the simpler methods and most of the tabular data is restricted to a fire exposure corresponding to the standard fire curve.

Whatever model is adopted the analysis needs to consider transient behaviour which covers:

- Heat transfer within the element including conduction for solid elements but also any radiative or convective components particularly where the construction includes cavities and/or voids.
- Moisture migration.
- Chemical reactions and phase changes.

In order to undertake the analysis, knowledge of material properties at elevated temperature is required specifically:

- thermal conductivity;
- specific heat;
- density;
- emissivity;
- initial moisture content;
- charring rate if appropriate.

As the guidance in this manual is aimed principally at practising structural engineers the fundamental theory is not considered and the focus is on tabulated data and simple calculation

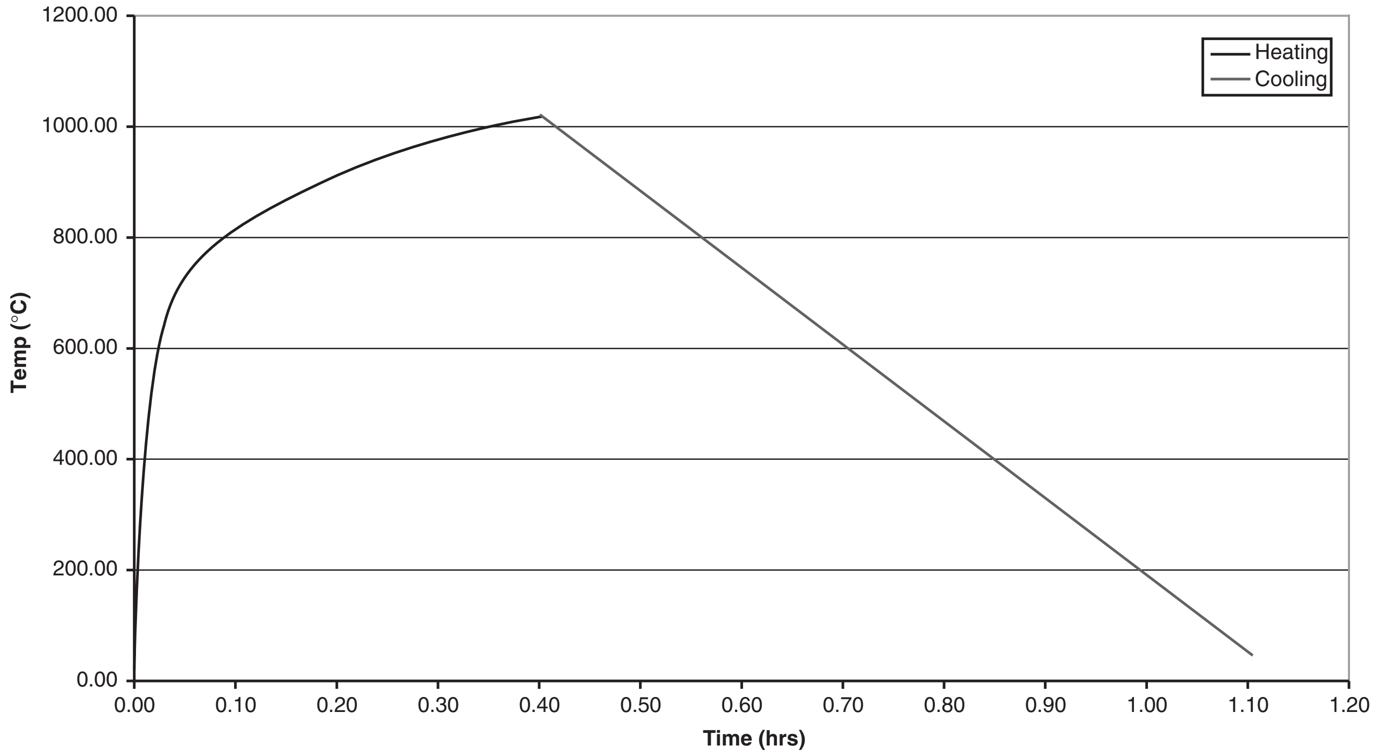

Figure 11.6 Parametric curve

models. The structural Eurocodes provide methods for determining temperature distributions subject to certain conditions. The thermal modelling approaches set out in the Eurocodes are summarised in **Table 11.2**.

Heat transfer methods for materials that incorporate free moisture should consider the effect of moisture migration with time through the member in order to provide an accurate prediction of the temperature of the element with time. This is generally accomplished through the incorporation of mass transfer in the model providing additional information on the pressure field due to steam production which, in certain cases, may influence the tendency of a material to spalling. For many simple models, the influence of moisture is either implicitly included (empirical models and tabulated data) or conservatively ignored.

Eurocode	Material	Tabular data	Simple model	Advanced model
EN 1992-1-2	Concrete	Yes	No	Yes
EN 1993-1-2	Steel	No	Yes	Yes
EN 1994-1-2	Composite (steel and concrete)	Yes	Yes	Yes
EN 1995-1-2	Timber	No	Yes	No
EN 1996-1-2	Masonry	Yes	Yes	Yes
EN 1999-1-2	Aluminium	No	Yes	Yes

Table 11.2 Thermal modelling options in the structural Eurocodes

11.3.1 Concrete

For materials with a high thermal conductivity (such as steel) it is generally possible to ignore thermal gradients within the member and assume a uniform temperature. However, for concrete members having a low thermal conductivity and including free and chemically bound moisture, the calculation of heat transfer to the structure can be very complex. A number of different methods may be used to derive the temperature distribution within the member. Eurocode 2 includes a number of temperature profiles for slabs, beams and columns with the temperature profile for slabs also being applicable to walls subject to heating from one side. The temperature profiles are presented for specific fire resistance periods and are therefore applicable only to a heating regime corresponding to a standard fire exposure. In principal, the calculation methods for which the temperature profile is input data could be used to determine performance due to different thermal exposure but there are no validated test data to support this.

11.3.2 Structural steel

Steel loses both strength and stiffness with increasing temperature. It should be borne in mind that the determination of strength reduction factors for hot rolled steel is dependent not only on the material but also on the test method, the heating rate and the strain limit used to determine steel strength. The differences between test data are significant. The British

Steel data used in the National and European codes show that for a temperature of 550°C structural steel will retain 60% of its room temperature strength while the corresponding figure obtained from the ECCS relationship for the same temperature is closer to 40%. The use of the British Steel data is justified by their improved correlation with large-scale beam and column tests, both in terms of the heating rates and the strains developed at the deflection limits imposed by the standard fire resistance tests. This simplified presentation does not itself take into consideration the fact that values above unity exist within the lower range of temperatures. The fine detail in the temperature-dependent material properties is principally of interest to those involved in the numerical modelling of material and structural behaviour. What is abundantly clear is that both strength and stiffness decrease with increasing temperature and that this reduction is particularly significant between 400 and 700°C.

Because of the perceived poor performance of steel elements in fire discussed above, the most common method of 'designing' for fire is to design the steel structure for the ambient temperature loading condition and then to protect the steel members with proprietary fire protection materials to ensure that a specific temperature is not exceeded or, in the light of the discussion above, that a specified percentage of the ambient temperature loading capacity is retained.

Traditional fire design methods for structural steel are based on the concept of a single 'critical' temperature. Due to the relationship between steel strength and temperature the figure of 550ºC is generally adopted as the critical temperature for steel. In reality there is no single critical temperature as the capacity of the structure is a function of the load applied at the fire limit state. This is discussed further in the section dealing with the calculation of the mechanical response of structural elements.

The rate of increase in temperature of a steel cross-section is determined by the ratio of the heated surface area (A) to the volume (V). The ratio A/V is known as the section factor and is analogous to the earlier concept whereby the rate of temperature rise was related to the ratio of the heated perimeter (H_p) to the area of the section (A). A steel section with a large surface area will be subject to a greater heat flux than one with a smaller surface area. The greater the volume of the section the greater will be the heat sink effect. Therefore, a small thick section (such as a UC section) will heat up to a given temperature more slowly than a long thin section. In terms of applying passive fire protection the greater the section factor the greater the thickness of protection required to limit the temperature of the steel to a given temperature.

The most common method used in the UK to relate protection thickness to section factor for a given fire resistance period and a specified critical temperature is the 'Yellow Book' published by the Association for Specialist Fire Protection (2007).

The European fire design standard for steel structures includes methods for calculating the temperature rise in both unprotected and protected steel assuming a uniform temperature distribution through the cross-section. The increase of temperature $\Delta\theta_{a,t}$ for an unprotected member during a time interval Δt is given by:

$$\Delta\theta_{a,t} = k_{sh}\frac{A_m / V}{c_a\rho_a}h_{net,d}\Delta t \quad \text{for } \Delta t \le 5\,\text{sec} \tag{11.10}$$

where

ρ_a	is the unit mass of steel [kg/m^3];
A_m	is the surface area of the member per unit length [m^2];
A_m/V	is the section factor for unprotected steel members [m^{-1}];
c_a	is the specific heat of steel [J/kgK];
$\dot{h}_{net,d}$	is the net heat flux per unit area [W/m^2];
k_{sh}	is correction factor for the shadow effect (k_{sh} = 1.0 if the shallow effect is ignored);
Δt	is the time interval [seconds];
V	is the volume of the member per unit length [m^3].

For circular or rectangular cross-sections fully engulfed by fire the shadow effect is not relevant and k_{sh} = 1.0 otherwise: for I sections under normal file actions for the other cases

$$k_{sh} = \begin{cases} \dfrac{09[A_m / V]_b}{A_m / V} \\ \dfrac{[A_m / V]_b}{A_m / V} \end{cases} \tag{11.11}$$

In the above equation the value of A_m/V should not be used if it less than 10 m^{-1}. $[A_m/V]_b$ is the box value of the section factor.

The k_{sh} correction for the 'shadow effect' accounts for the fact that members with geometry similar to I and H sections are shielded from the direct impact of the fire in some parts of the surface.

The above method requires integration with respect to time with the calculated temperature rise substituted back into the equation for each time step. This can be realised using a simple spreadsheet based method. For greater accuracy temperature-dependent values for specific heat and thermal conductivity could be used (where known).

For protected members a similar procedure is adopted taking into account the relevant material properties of the protection material. The method is applicable to non-reactive fire protection systems such as board or spray protection but is not appropriate for reactive materials such as intumescent coatings. Assuming a uniform temperature distribution the temperature rise $\Delta\theta_{a,t}$ of a protected steel member during a time interval Δt is given by:

$$\Delta\theta_{a,t} = \frac{\lambda_p A_p / V}{d_p c_a \rho_a} \frac{(\theta_{g,t} - \theta_{a,t})}{(1+\phi/3)} \Delta t - (e^{\phi/10} - 1)\Delta\theta_{g,t} \qquad (11.12)$$

With $\Delta\theta_{a,t} \geq 0$ and $\phi = \frac{c_p \rho_p}{c_a \rho_a} d_p A_p / V$

where

- λ_p is the thermal conductivity of fire protection material [W/mK];
- $\theta_{a,t}$ is the steel temperature at time t [°C];
- $\theta_{g,t}$ is the ambient gas temperature at time t [°C];
- $\Delta\theta_{g,t}$ is the increase of ambient gas temperature during time interval Δt [K];
- ρ_a is the unit mass of steel [kg/m^3];
- ρ_p is the unit mass of fire protection material [kg/m^3];
- A_p/V is the section factor for steel members insulated by fire protection material [m^{-1}];
- A_p is the appropriate area of fire protection material per unit length [m^2];
- c_a is the temperature-dependent specific heat of steel [J/kgK];
- c_p is the temperature-independent specific heat of fire protection material [J/kgK];
- d_p is the thickness of fire protection material [m];
- Δt is the time interval [seconds];
- V is the volume of the member per unit length [m^3].

11.3.3 Composite steel and concrete construction

The European fire design standard for composite construction provides a conservative estimate of the temperature rise in composite slabs through tabulated data treating the composite slab as if it were a solid slab. The temperatures at a distance x from the underside of the exposed slab are related to specific standard fire resistance periods in **Table 11.3**.

11.3.4 Timber and masonry

In general there is no need to determine the temperature distribution through a timber structural element as capacity is related to a residual undamaged section below the char layer where the material is assumed to maintain its ambient temperature properties in terms of strength and stiffness. The important aspect in this case is the calculation of the depth of charring.

The fire part of Eurocode 6 provides tables of minimum dimensions to achieve specified periods of fire resistance; it also includes time-temperature graphs for various fire resistance periods for different types of masonry. For insulation purposes the calculation of the temperature rise of the unexposed face is reasonably well understood and the Eurocode includes temperature-dependent material properties for use in thermal modelling. However, the issue of free and chemically bound water needs to be addressed to be able to accurately reflect the delay in reaching temperatures significantly above 100ºC. Other issues that need to be considered include the presence of voids in hollow masonry blocks and ancillary products (such as metal wall ties) leading to localised areas of high conduction.

11.3.5 Aluminium

Although not readily associated with fire resistant structural design, BS EN1999-1-2 provides guidance on the use of simple and advanced calculation models for aluminium structures subject to fire. The code effectively utilises many of the procedures set out in BS EN1993-1-2 in terms of the calculation of heat transfer to external members (Annex B), and in the verification methods related to aluminium temperature development and calculation of the resistance of cross-sections. The most significant difference between the two codes is that the thermal and structural material property data only extend up to 500ºC at which point the strength and stiffness of aluminium is zero. The reduction in strength with temperature for aluminium depends on the specific alloy adopted. **Figure 11.7** illustrates the lower range of values for the 0.2% proof strength ratios for the alloys covered in the Eurocode.

Depth x (mm)	Temperature θ_c (ºC) for standard fire resistance of R30	R60	R90	R120	R180	R240
5	535	705				
10	470	642	738			
15	415	581	681	754		
20	350	525	627	697		
25	300	469	571	642	738	
30	250	421	519	591	689	740
35	210	374	473	542	635	700
40	180	327	428	493	590	670
45	160	289	387	454	549	645
50	140	250	345	415	508	550
55	125	200	294	369	469	520
60	110	175	271	342	430	495
80	80	140	220	270	330	395
100	60	100	160	210	260	305

(Note: for lightweight concrete the values may be reduced to 90% of those given)
For the temperature of the reinforcement and the temperature of the steel decking the Eurocode presents a method based on the use of coefficients to determine the temperature for specific periods of fire resistance.

Table 11.3 Temperature distribution in a solid normal weight concrete slab of 100 mm thickness. Data taken from BS EN 1994-1-2. Permission to reproduce extracts is granted by BSI

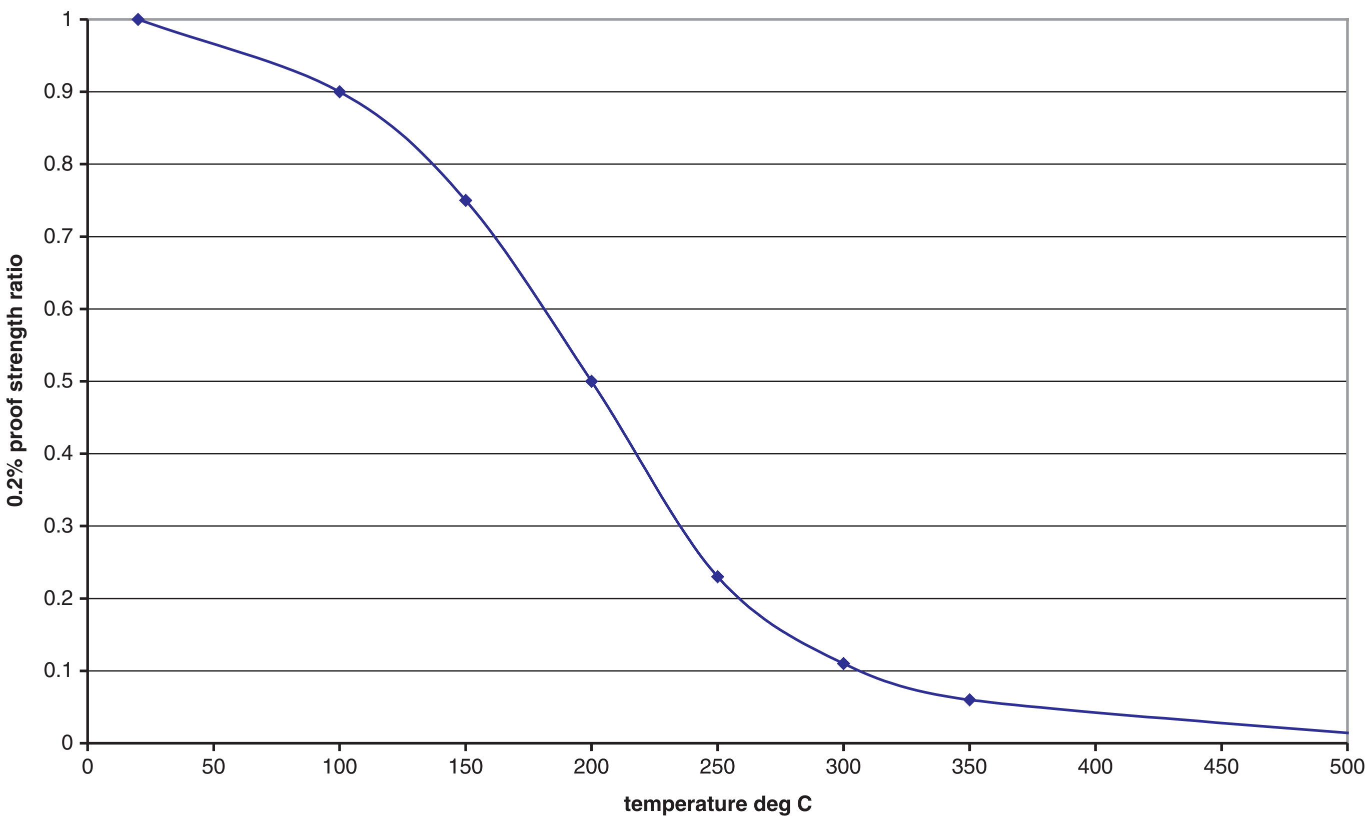

Figure 11.7 0.2% strength ratios (lower limits) for aluminium alloys

11.4 Mechanical (structural) response

Once the thermal analysis has been carried out to ascertain the compartment atmosphere temperatures and the heat transfer to the structure has been completed it is then necessary to assess the effect of the increased temperatures on the resistance of the structural members. In reality, steps 2 and 3 of the fire engineering design (heat transfer and structural response) will generally be undertaken in tandem, with the rules for calculating or looking up member temperatures within the same standards as the rules for evaluating member capacities.

The most comprehensive suite of design standards for undertaking structural fire engineering design are the structural Eurocodes. The fire codes cover actions on structures exposed to fire as well as design procedures for concrete, steel, composite steel and concrete, timber, masonry and aluminium. All these codes have now been published by BSI for use in the UK along with a National Annex setting out Nationally Determined Parameters for those areas where National choice is allowed. Before looking at the methods for determining structural response it is necessary to look at the relationship between design loading at ambient temperature and the design load case for the ultimate limit state for the accidental design situation of a fire. This is the subject of the next section.

11.4.1 Load effects at the fire limit state

Traditional design procedures for steel structures are based on limiting the temperature rise of the steel section to a set value generally termed the 'critical' temperature for steel. Similarly tabulated values in the National code for the fire design of concrete structures specify minimum cover distances to ensure that the temperature of the reinforcement does not exceed a specified limiting value. Such methods are independent of the load applied under fire conditions and offer simplified often conservative solutions to the majority of fire design scenarios.

The development of structural fire engineering has highlighted the importance of load in determining the fire resistance of structural elements. A major change in the design methodology for steel structures in fire came about with the publication in 1990 of BS 5950 Part 8. Although this code is still based on an evaluation of the performance of structural steel members in the standard fire test it allows architects and engineers an alternative approach of designing for fire resistance through calculation procedures. It recognises that there is no single 'failure temperature' for steel members and that structural failure is influenced not only by temperature but also by load level, support conditions and the presence or otherwise of a thermal gradient through and/or along the member. The code allows for the consideration of natural fires but does not provide any detailed information or guidance. Load factors and material strength factors specific to the fire limit state are given. These are partial safety factors which deal with the uncertainties inherent in probabilistic distributions for loading and material properties and represent reductions from ambient temperature design in recognition of the small probability of excessive loads being present at the same time as a fire occurs. In 2003, BS5950 Part 8 was updated to provide consistent information with the fire part of Eurocode 3.

The national code for the design of concrete structures, BS 8110 Part 2, did not reflect the important role that load level plays in determining performance under fire conditions. Load effects are allowed for in Eurocode 2 for the tabulated data for concrete structures with dimensions dependent on load level for columns and load-bearing walls.

An accurate assessment of the performance of a structural member during a fire requires knowledge of both the reduction in material properties with increasing temperature and an accurate assessment of the loads acting on the structure at the time of the fire. Load effects can have a significant impact on the fire resistance of a structure and this is reflected in the requirement for realistic load levels to be in place during standard fire tests. As material properties reduce with increasing temperature the load-bearing failure criterion is reached when the residual strength of the element equals the load applied. Load level can also have a significant impact on other types of construction such as timber or light steel framing that rely on sacrificial linings for fire resistance. Increased loading leads to increased deflections at the fire limit state which can cause gaps to open between panels thereby compromising the assumed level of fire protection.

Loads are factored and a number of load cases considered for the ambient temperature situation to account for uncertainties and the potential for adverse conditions. Fire in terms of the Eurocode system is an ultimate limit state accidental action and, as such, is subject to specific partial factors that reflect the reduced likelihood of the full ambient temperature design loading being present at the same time as a fire occurs. In the European system in order to determine the calculation of the load effects at the fire limit state the designer must be familiar with the Basis of Design EN 1990 which provides the required load combinations and with the fire part of the Eurocode for Actions on Structures EN 1991-1-2 which, in addition to specifying the fire design to be adopted also specifies the mechanical actions for structural analysis. In particular, EN 1991-1-2 specifies the partial factor for imposed (assuming leading variable action) loading for the fire limit state. Fire loading is an ultimate limit state accidental design situation of the form:

$$E_d = E\,(G_{k,j};\, P;\, A_d;\, (\Psi_{1,1} \text{ or } \Psi_{2,1})Q_{k,i}) \quad \text{for } j \geq 1;\, i > 1 \tag{11.13}$$

where

E the effect of actions (E_d is the design value of the effect of actions)

G permanent action (dead load)

P relevant representative value of a pre-stressing action (where present)

A_d design value of an accidental action

Ψ_1 factor for frequent value of a variable action

Ψ_2 factor for quasi-permanent value of a variable action

Q_k characteristic value of a single variable action ($Q_{k,1}$ is the characteristic value of the leading variable action – often the imposed load)

In the fire situation, A_d is the effect of the fire itself on the structure, i.e. the effects of restrained thermal expansion, thermal gradients, etc. However, where the design is based on the standard fire situation then such indirect actions need not be considered.

EN 1990 allows the use of either Ψ_1 or Ψ_2 with the main variable action. EN 1991-1-2 recommends the use of Ψ_2. However, the UK National Annex for use with EN 1991-1-2 specifies Ψ_1 to be used in the UK. The value of the partial factors for specific types of occupancy and design situations is shown in **Table 11.4**.

It is important to understand the significance of the reduced partial factor for imposed loading and the effect that this has on different structural forms. Effectively a reduction in the imposed load will increase the fire resistance of the structural member. Consequently those forms of construction where the imposed load is a relatively high proportion of the total load (such as steel frame construction) may be able to reduce the levels of fire protection required by taking advantage of the spare capacity in the member. Conversely for those forms of construction (such as reinforced concrete) where the imposed load is a relatively small proportion of the total load the potential benefits of a fire engineering solution taking into account residual capacity are limited. The relationship between the

Action	Ψ_1	Ψ_2
Imposed loads in buildings	0.5	0.3
Category A: domestic, residential	0.5	0.3
Category B: office areas	0.7	0.6
Category C: congregation areas	0.7	0.6
Category D: shopping areas	0.9	0.8
Category E: storage areas	0.7	0.6
Category F: traffic area, ≤ 30 kN	0.5	0.3
Category G: traffic area, 30–160 kN	0	0
Category H: roofs		
Snow load: H ≤ 1000m a.s.l.	0.2	0
Wind loads on buildings	0.2	0

Table 11.4 Values of partial factors (Ψ_{fi}) to be used for the accidental fire limit state. Data taken from BS EN 1990. Permission to reproduce extracts is granted by BSI

reduction factor η_{fi} and the ratio of the dead and imposed loads is illustrated in **Figure 11.8** where:

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}} \tag{11.14}$$

with:

$Q_{k,1}$ = characteristic value of the leading variable action (imposed load)

G_k = characteristic value of a permanent action (dead load)

γ_G = partial factor for permanent actions (1.35)

$\gamma_{Q,1}$ = partial factor for variable action 1 (1.5)

ψ_{fi} = combination factor (= 0.5 for residential and office applications from UK National Annex to EN 1991-1-2)

11.4.2 Calculation methods

A number of calculation methods are available ranging from simple tabulated data through to advanced numerical methods. Advanced numerical methods which consider nonlinear behaviour at elevated temperature require specific areas of expertise and in general would not be available to practising structural engineers. The fire parts of the structural Eurocodes include tabulated data and simplified calculation methods which can be used by engineers familiar with ambient temperature design procedures. The nature of the calculation procedures is determined in part by the current state of knowledge with respect to the behaviour of the specific construction materials at elevated temperature. However, there are some common principles that apply to all materials. Simple calculation methods are based on:

- A knowledge of the design procedures at ambient temperature.
- An understanding of the partial factors for load effects to be used at the fire limit state.

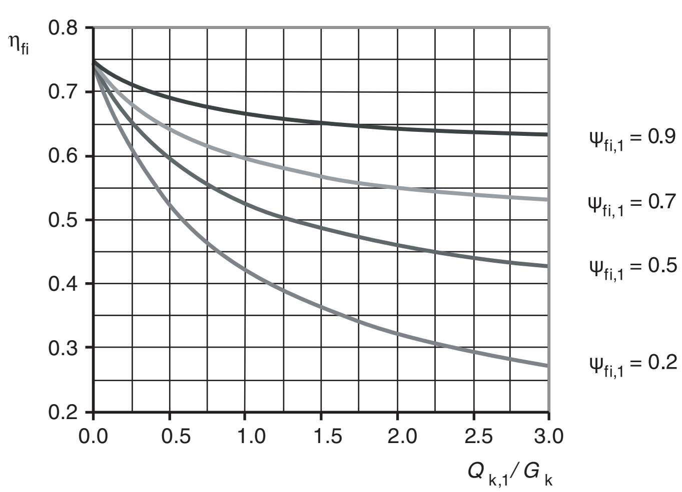

Figure 11.8 Relationship between reduction factor η_{fi} and ratio of dead and imposed loads for values of the partial factor for the fire situation ψ_{fi}

- A knowledge of the reduction in material strength and stiffness at elevated temperature and familiarity with reduction factors to be used for given temperatures.

A detailed breakdown of the various calculation methods available is beyond the scope of this publication.

11.5 Conclusion

Many structural engineers will be unfamiliar with the principles of structural fire engineering design. In recent years, a number of specialist consultants have emerged offering fire engineering solutions, largely for prestigious projects where the potential benefits of adopting a fire engineering design approach outweigh the additional design cost to the client. There is a fundamental lack of understanding of the principles of structural fire engineering design. In reality, the design methodology, as set out in the fire parts of the structural Eurocodes, is based on the principles adopted for normal temperature design. One of the aims of this simplified guidance is to demystify the subject so that it can be readily understood and used by structural engineers familiar with the underlying principles and assumptions of design for the ambient temperature condition.

11.6 References

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Chapter 12

Structural robustness

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The design of a building to be robust is an essential part of good structural design. Nominally, buildings may be stable under normal actions but abnormal actions – such as accidental damage, unexpected structural movement or the effects of poor quality control in construction – can produce consequences that are disproportionate to the initial event. The design of buildings to be robust – to ensure that the extent of structural damage is in line with the scale of the assault – has been a central facet of UK design since the late 1960s, and has now been embedded in the Eurocodes. Similar, often more limited, measures have been implemented elsewhere around the world. This chapter explains the basis of design for structural robustness, gives practical guidance to the engineer undertaking a design against disproportionate collapse in accordance with the UK Building Regulations and Eurocodes, and discusses some of the issues that need to be considered when designing for robustness.

CONTENTS

12.1 Introduction

Structural robustness is a quality of a structural system which describes an ability to withstand in a proportionate manner local damage to which the building might reasonably be subjected. Usually this is defined as the avoidance of an escalation of the consequences of the local damage into a more widespread collapse. Robustness enables a structural system to withstand a degree of damage from an action which is beyond the design basis of the structural design, but is nevertheless significant enough for the limitation of damage and the risks to the building occupants or users to be of value.

All structural systems pose an associated risk to the users or occupants of that structure, whether through error in design, substandard material quality or errors during construction, or structural failure resulting from loads exceeding the design basis. It is the aim of the structural designer to mitigate the risks to the users of the structure so far as reasonably practicable. Quality assurance during design and construction is one aspect of this mitigation, while the designer takes reasonable steps to calculate loads that will not be exceeded during the design life of the structure, principally through the use of material and load partial factors and a limit states design approach which seek to achieve a tolerably low probability of failure of the structural system.

There does, however, remain a risk that a structural element will fail during the design life of the structure. Similarly, while the designer seeks to ensure that the design loads will not be exceeded, there is a risk that the structure will be subjected to an unforeseen action against which it is not specifically designed. Robustness is the ability of the structural system to limit the consequences of the abnormal failure of a structural element, or to limit the damage caused by an unforeseen action against which it has not been specifically designed.

Through the ability to limit the consequences resulting from damage on a local scale, robustness therefore plays an important part in reducing the risks associated with the structural system as far it is reasonably practicable to do so and is an indisputably desirable quality in structural design. Buildings in the United Kingdom are rarely designed against seismic loads due to the low geographical seismic hazard, and the structural columns of a framed building may not be specifically designed to withstand the impact of an errant vehicle or the blast load from a terrorist attack. Nevertheless, a robust building will be better able to withstand the demands arising from such hazards than a building designed merely for the design basis loads with no consideration of beyond-design basis events.

Structural robustness is a form of ductility: a ductile response in a material or structural system is one in which the system can undergo significant deformation without suffering loss of strength and in which failure is gradual and predictable, in contrast to a brittle response in which the onset of deformation results in a rapid loss of strength and sudden failure. A robust structure is one in which at the system level it is able to sustain damage and in which eventual failure is gradual and predictable.

12.2 Disproportionate and progressive collapse

Design for robustness in a structural system is closely associated with design against both disproportionate collapse and progressive collapse. The terms are often used interchangeably or erroneously, and clarification of the terms is necessary.

Structural robustness is a quality in a structure of insensitivity to local damage, in which modest damage (irrespective of whether of a foreseeable or unforeseeable action) causes only a similarly modest change in the structural behaviour. More specifically, a robust structure has the ability to redistribute load in the event that a load-bearing member suffers a loss of strength or stiffness, characteristically exhibiting a ductile rather than a brittle global response. Robustness does not mean a structure is overdesigned: the ability to resist damage is achieved through consideration of the global structural behaviour and failure modes so that the effects of a localised structural failure can be mitigated by the ability of the structure to redistribute the load elsewhere, and so that the effects of the initial failure are gradual in onset.

Eurocode 1 (BS EN1991-1-7) (BSI, 2006) describes robustness as 'the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error without being damaged to an extent disproportionate to the original cause', thereby linking it explicitly to the concept of disproportionate collapse while recognising that total collapse is an acceptable outcome from a gross hazard.

A *disproportionate collapse* is one which is judged (by some measure defined by the observer) to be *disproportionate* to the initial cause. This is merely a judgement made on observations of the consequences of the damage which results from the initiating events and does not describe the characteristics of the structural behaviour. In contrast, a *progressive collapse* is one which develops in a *progressive* manner akin to the collapse of a row of dominos, successive alternative loadpaths becoming overloaded and successively failing (Arup, 2011). A collapse may be progressive horizontally – successively from one structural bay to those adjacent to it and propagating through the structural frame. A collapse may also be progressive vertically, for example, the collapse of the columns supporting a floor slab due to the dynamic shock load caused by the collapse onto it of the storey above, or the successive collapse of the columns supporting a number of floors due to the dynamic shock load as the block of mass is brought to rest as it impacts with more rigid structure below. These examples of vertical progressive collapse are often termed 'pancaking' (downward and upward respectively). The term *progressive* refers to a characteristic of the behaviour of the structural collapse.

A collapse may be *progressive* in nature but not necessarily *disproportionate* in its extents, for example, if arrested after it progresses through a number of structural bays. Vice versa, a collapse may be *disproportionate* but not necessarily *progressive* if, for example, the collapse is limited in its extents to a single structural bay but the structural bays are large.

12.3 Basic approaches to design for robustness

There are two basic approaches to designing for robustness. *Prescriptive approaches* are those in which the measures to be taken by the designer and the forces for which the structure is to be designed are prescribed in codes of practice, standards or good-practice references. The underlying structural theory and basis of the measures prescribed may not be evident, the structure deemed to satisfy the requirements provided all necessary measures have been complied with. Such approaches, termed *indirect design* in the US, include tie-force design, in which the designer is required to ensure the structure is capable of resisting minimum tie forces, whether peripheral, transverse or vertical. The problems of such prescriptive or *rule-based* approaches are twofold: firstly that because the underlying structural theory is not necessarily evident to the designer it may be incorrectly applied or applied in inappropriate circumstances which were not necessarily envisaged when the measures were set down; and secondly that the level of robustness achieved is not usually quantifiable: the designer cannot state that the structure has achieved any particular level of robustness or reduced the risks to users of the structure to a particular threshold, but merely that the design complies with the code requirements.

The second approach to designing for robustness is a *quantitative approach*, in which the designer is required to demonstrate through structural analysis that the structure can achieve a certain performance criterion such as the ability to withstand the loss of a column, beam or length of load-bearing wall without resulting in a collapse greater than a given area. Rather than requiring the structure to be designed for a particular force as in a prescriptive approach, the structure is required to achieve a particular level of performance and in the design process the engineer calculates the loads to which the structure might be subjected under the scenario specified. As such, it is a *performance-based* approach and as a minimum requires definition of both the design scenario (e.g. notional removal of a column) and the performance condition to be achieved (e.g. collapse limited to 100 m²). It is directly comparable with performance-based design used in seismic or fire engineering: the design scenario (e.g. a one in 475-year earthquake) and the performance condition (e.g. life safety) are again specified. (A 475-year return period corresponds to a probability of exceedance in a 50-year design life of 10% ($= 1 - (1 - {}^{1}/_{475})^{50}$) (Booth *et al*., 2006).) A performance-based approach, termed a *direct design* approach in the US, permits demonstration that the structural system has achieved a particular level of robustness rather than merely complying with code requirements; however, it is analytically more complex and potentially open to greater subjectivity than a prescriptive approach.

12.4 Historical development of design for structural robustness

The first steps to implementing structural robustness requirements in national codes and standards for structural design were taken in the United Kingdom in the wake of the collapse of the tower block at Ronan Point in 1968 (Ministry of Housing and

Local Government, 1968b). A gas explosion in the kitchen on the eighteenth floor of a tower block at Ronan Point caused a progressive collapse of the corner of the building due to the failure of the structural precast cladding panels (See Chapter 9: *How Buildings Fail*). The cladding was incapable of redistributing the gravity loads from the structure above after the blast loads caused failure of the cladding panels on the explosion floor, and the collapse propagated over almost the full height of the tower.

The inquiry into the collapse led to recommendations being issued by the Ministry of Housing and Local Government (Ministry of Housing and Local Government, 1968a) that large-panel structures should be built with alternative paths of support and exhibit stability against forces 'liable to damage the load-supporting members', which could be assumed to be equivalent to 5 psi (34 kPa). This load was derived from observational and experimental evidence of the estimated failure load of the wall panel at Ronan Point whose failure initiated the collapse. The recommendations were incorporated into the Fifth Amendment of the Building Regulations (1970) and applied to all buildings having five or more storeys, and required that if any structural member were to be removed, failure should not exceed 750 square feet (70 square metres) or 15% of the area of the affected storey, whichever is the less, and would not extend beyond the affected storey, and the storeys immediately above and below (**Figure 12.1**). In this requirement, the structural member was defined as a column or beam between adjacent supports (or between a support and the extremity of the member), or in the case of a load-bearing wall a length equivalent to 2.25 times the storey height. It was unnecessary for the design of a structural member to meet the requirements above if it was capable of sustaining a load of 5 psi (34 kPa) in any direction applied simultaneously with the combined dead and imposed load.

Prescriptive methods were first proposed as an alternative to the quantitative approach given in the Fifth Amendment by the Institution of Structural Engineers (1971), advocating that multi-storey framed structures in reinforced concrete or structural steel (as distinct from large-panel structures such as that at Ronan Point) were able to accommodate the loads envisaged in the Fifth Amendment provided the building satisfied the then-current British Standards and Codes of Practice and incorporated minimum levels of tying.

The current design requirements for England and Wales are a gradual evolution of those in the Fifth Amendment and are described Approved Document A of the Building Regulations (Office of the Deputy Prime Minister, 2004) and incorporated in the Eurocodes (Eurocode 1, BS EN1991-1-7:2006) (BSI, 2006). The Eurocodes formally became the national standards for the United Kingdom on 1 April 2010 and are used across Europe and elsewhere around the world. In the UK, the Eurocodes are complementary to Approved Document A and supersede the British Standards.

Requirements in other parts of the world vary in the level of detail defined, but where given are typically modelled on the UK approach. Inherent in design in countries such as Singapore, Hong Kong and Malaysia that adopt the British Standards is the implementation of the robustness requirements in each Code of Practice, though typically there is no document equivalent to Approved Document A which defines the national requirement,

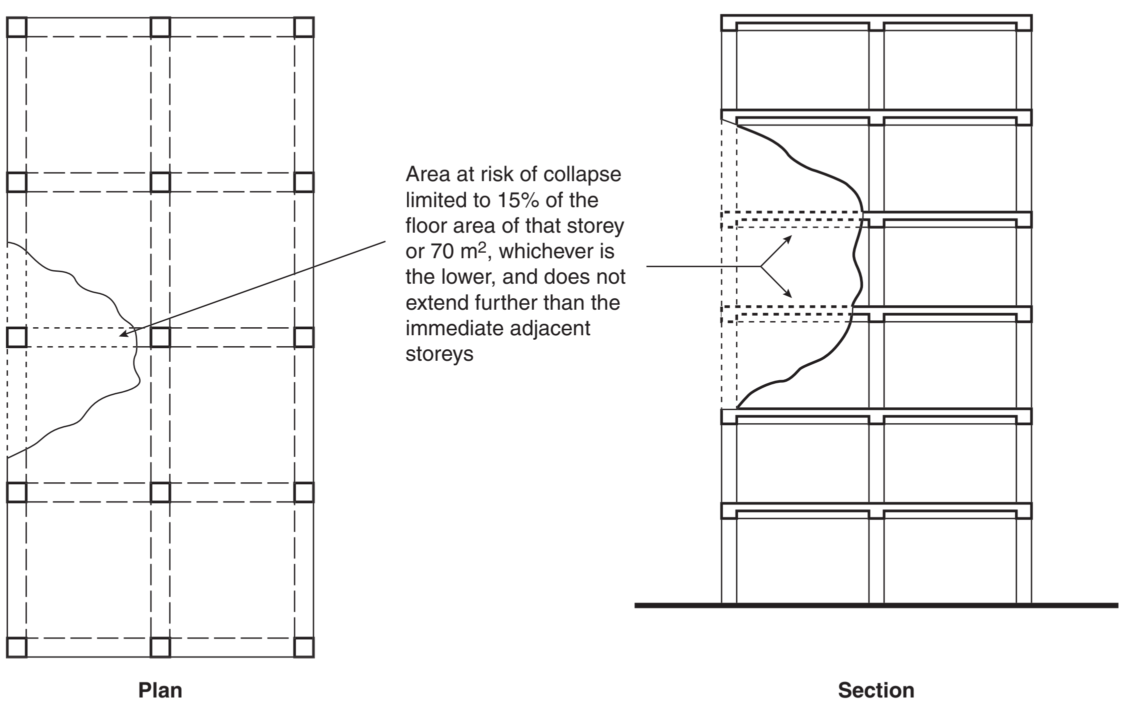

Figure 12.1 Area at risk of collapse in the event of an accident (Office of the Deputy Prime Minister, 2004) © Crown Copyright 2004

and the designer must therefore be aware of the differences that exist between the individual material codes.

In the United States, historically the requirement for design against disproportionate collapse has been minimal, limited to the high-level requirement that a building should be designed to sustain local damage with the structural system as a whole remaining stable and not being damaged to an extent which is disproportionate to the original local damage. Individual State or City building codes are free to define their own requirements, and the New York City Building Code quickly implemented measures similar to the United Kingdom (City of New York, 1973).

Elsewhere in the US, the introduction of design requirements for structural robustness was only triggered for federal and defence buildings by the collapse of the Alfred P. Murrah building in Oklahoma in 1995, and much more recently, the International Building Code introduced prescriptive measures for civilian buildings modelled on UK requirements in the 2009 edition of the code (International Code Council, 2009). For Department of Defense buildings, UFC 4–023–03 (United States Department of Defense, 2010) outlines a systematic design approach based on prescriptive tying, alternative load-path analysis and key element design according to the building risk classification which is closely modelled on and an enhancement of the approach defined in Approved Document A.

12.5 UK/European regulations and codes of practice

The requirements for design of structures in the UK for robustness are defined in Eurocode 1 (Annex A of BS EN1991-1-7:2006) (BSI, 2006). The Annex is informative rather than normative, the legal requirements being those set down in the Building Regulations for England and Wales, Scotland and Northern Ireland, each of which contain a similarly worded and broad requirement that a building 'shall not be constructed so that in the event of an accident the building will not suffer collapse to an extent disproportionate to the cause' (2004, 2010a, 2010b). More detailed guidance is given in Approved Document A (Office of the Deputy Prime Minister, 2004) for England and Wales and in similar publications for Scotland and Northern Ireland (Building Control Northern Ireland, 2009; Scottish Building Standards Agency, 2010a, 2010b) which describe a number of building risk classes and sets out the rules (design requirements) for each, and a critical appraisal of the requirements set down through the Building Regulations is given by Arup (2011). The Approved Documents contain official guidance and are not mandatory, having the same status as codes of practice in the United Kingdom. However, the Eurocodes and the British Standards which preceded them have been written to comply with the guidance given in the Approved Documents and compliance with the Approved Documents is normal in design except for structures which are not easily classified using the building risk classes given in the Approved Document or to which the rules given for the relevant risk class do not readily apply. In such instances, it is normal to ensure the level of robustness achieved in the design is at least equal to that implied by the Approved Document.

The requirements given in Annex A of BS EN1991-1-7:2006 are closely modelled on those in Approved Document A and the corresponding publications for Scotland and Northern Ireland, and it is anticipated that Approved Document A will be updated to reference the Eurocodes as approved standards for design. While the Annex is officially informative, it is to all intents and purposes rendered normative by the UK National Annex to BS EN1991-1-7:2006, which states that the 'guidance … should be used in the absence of specific requirements in BS EN1992-1-1 to BS EN1996-1-1 and BS EN1999-1-1 and their National Annexes'. Consequently it is anticipated and expected that requirements given in Annex A will be adopted as standard practice in Eurocode design.

12.6 Building risk class and design requirements

The design requirements for structural robustness contained in Annex A of BS EN1991-1–7:2006 are based on the building risk classes given in **Table 12.1**.

12.6.1 Class 1

For single-occupancy houses not exceeding four storeys, agricultural buildings and unoccupied buildings into which people rarely go, no specific measures are deemed necessary provided the building has been designed and constructed in accordance with the rules given in BS EN1990 to BS EN1999.

12.6.2 Class 2A

All other buildings are categorised as Class 2A or higher, for which the provision of effective horizontal ties or effective anchorage of suspended slabs to walls is required.

12.6.3 Class 2B

Hotels, flats, apartments and other residential buildings with greater than four storeys, office buildings with greater than four storeys, retailing premises with greater than three storeys, hospitals with three storeys or fewer, educational buildings with greater than one storey, car parks, and buildings to which the public are admitted and containing floor areas exceeding 2000 m² at each storey are categorised as at least Class 2B. For Class 2B buildings, either horizontal and vertical ties are required, or the building should be checked to ensure that upon the notional removal of each beam, column or nominal section of load-bearing wall, the building remains stable and the area of floor at risk of collapse does not exceed the smaller of 15% of the storey area or 100 m², and does not extend further than the immediate adjacent storeys (**Figure 12.1**). Where the notional removal of such an element would result in an extent of damage in excess of this limit, it should be designed as a 'key element'. It should be noted that while the wording of the current edition of Approved Document A3 does not require horizontal ties to be provided if the alternative loadpath approach is adopted, horizontal ties *should* always be provided in Class 2B buildings regardless of whether vertical ties or alternative loadpath analysis is used, unless there are clearly justifiable reasons why this should not be the case. The potential for the situation to arise whereby the robustness of a Class 2B building may be permitted to be less than that of otherwise similar Class 2A building is clearly unjustifiable.

12.6.4 Class 3

Car parks with more than six storeys, hospitals with more than three storeys, and all other buildings exceeding fifteen storeys, or to which the public are admitted and containing floor areas greater than 5000 m² at each storey, are categorised as Class 3, together with all buildings to which the public are admitted in significant numbers, stadia accommodating more than 5000 spectators and buildings containing hazardous substances and/or processes. For Class 3 buildings, a systematic risk assessment of the building is required which takes into account both foreseeable and unforeseeable hazards.

While not specifically required by Approved Document A, it is indisputable that the design of a Class 3 building should meet the requirements of Class 2B as a minimum. There may be circumstances in which the application of the requirements for a Class 2B building is not straightforward, for example, in special structures, sculptures or structures that are not conventionally framed. In such cases, the design should demonstrate by alternative means that the structure is at least as robust as that implied by the requirements for Class 2B buildings in the context of framed construction.

In some circumstances it may be useful for the structural engineer to recommend the designation of buildings as Class 3 which are not formally classified as such. The structural engineer may explain to clients that it will rarely be detrimental to the design to do so, as a minimum ensuring that the risks to the building have been assessed in a systematic manner without necessarily making compliance with the Building Regulations more onerous. Examples of cases where such an approach may be useful are buildings which are high-value client assets, buildings for which the risk of terrorist attack is a foreseeable hazard, or buildings which serve a critical function either to the occupier or more widely. Where the risk assessment does result in the identification of particular measures the incorporation of which measurably reduces the risk associated with one or more hazards, the designation of the structure as a Class 3 building will have caused the risks to be better managed and the design to be more satisfactory as a result. However, even if the risk assessment concludes that no additional measures are required over and above those indicated by Approved Document A, the systematic risk assessment will have value in providing the client with an audit trail which demonstrates that all foreseeable risks have been rigorously identified and assessed.

12.7 Interpretation of building risk class and design requirements

The definition of the building risk class is usually straightforward, but in some instances may be open to interpretation, perhaps because the building's use is ill-defined, is mixed use, has a varying number of storeys, incorporates mezzanine floors, incorporates an unoccupied plant enclosure at roof level or due to the rules on the classification of basement storeys. For buildings undergoing alteration, modification or extension it is also sometimes unclear what building risk class and therefore design requirements should apply. The discussion below is written specifically with respect to the application of Approved Document A in the United Kingdom and is based on previous decisions of Building Control Officers on compliance with the Building Regulations. Similar issues apply in other jurisdictions although the thresholds may be different.

Consequence class	Example of categorisation of building type and occupancy
1	Single occupancy houses not exceeding 4 storeys. Agricultural buildings. Buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance of 1.5 times the building height.
2a Lower risk group	5 storey single occupancy houses. Hotels not exceeding 4 storeys. Flats, apartments and other residential buildings not exceeding 4 storeys. Offices not exceeding 4 storeys. Industrial buildings not exceeding 3 storeys. Retailing premises not exceeding 3 storeys of less than 1000 m² floor area in each storey. Single storey educational buildings All buildings not exceeding 2 storeys to which the public are admitted and which contain floor areas not exceeding 2000 m² at each storey.
2b Upper risk group	Hotels, flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys. Educational buildings greater than single storey but not exceeding 15 storeys. Retailing premises greater than 3 storeys but not exceeding 15 storeys. Hospitals not exceeding 3 storeys. Offices greater than 4 storeys but not exceeding 15 storeys. All buildings to which the public are admitted and which contain floor areas exceeding 2000 m² but not exceeding 5000 m² at each storey. Car parking not exceeding 6 storeys.
3	All buildings defined above as Class 2 Lower and Upper Consequences Class that exceed the limits on area and number of storeys. All buildings to which members of the public are admitted in significant numbers. Stadia accommodating more than 5000 spectators. Buildings containing hazardous substances and/or processes.

Note 1 For buildings intended for more than one type of use the 'consequences class' should be that relating to the most onerous type.
Note 2 In determining the number of storeys basement storeys may be excluded provided such basement storeys fulfil the requirements of 'Consequences Class 2b Upper Risk Group'.
Note 3 Table A.1 is not exhaustive and can be adjusted.

Table 12.1 Categorisation of consequence classes (BSI, 2006). Permission to reproduce extracts from BS EN 1991-1-7:2006 is granted by BSI

12.7.1 Basement storeys

Any basement storeys may be excluded when determining the number of storeys, provided the design of such basement storeys at least fulfil the requirements for Class 2B buildings. The definition of a basement storey is sometimes unclear: for example, when does a lower ground floor become a basement? In London and many other cities it is common for both domestic and commercial office buildings to be designed set back from the pavement to create a lightwell, with a habitable lower ground/basement floor. In such buildings there is often no structural difference between the storeys and there remains the potential for vehicle impact (**Figure 12.2**), and consequently there is no justification for the omission of such storeys from the determination of the number of storeys.

In contrast, if an office building has three storeys above ground, for example, but has several levels of plant in deep basements, there is no justifiable reason for it being subject to more onerous design requirements than a building that has only one basement level but is otherwise identical, unless the function of those basement levels itself introduces an additional hazard or increases an existing one, such as the provision of car parking (see below).

Figure 12.2 Risk of vehicle impact on lower ground floor. BARCROFT MEDIA

True basement storeys are relatively insulated from external hazards such as vehicle impact, and are often inherently sufficiently robust that the collapse of the superstructure above is unlikely to propagate downwards into the basement, the ground floor slab acting as a strong floor arresting the collapse. A building in which there is a fall in level across the site such that at the front of the building the ‘ground floor’ storey is at street level but at the rear of the building the ‘basement’ is at grade (perhaps enabling vehicle access to the building) would not be designed in the spirit of the requirements if the basement storey were excluded.

A typical example of a building in which designation of the basement as Class 2B and the subsequent exclusion of the basement storeys from the number of storeys on which the risk classification is based would be appropriate would be one where the basement is fully below ground and inaccessible around the full perimeter of the building. Where this is not the case, for example, where the building is set back from the pavement with a lightwell to the basement floor, risk of structural damage may exist due to hazards such as an errant vehicle falling into the lightwell. In such circumstances, it would be inadvisable to exclude the basement storey from the storey count.

Where exclusion of the basement levels from the storey count is justified, it is of course imperative that the designer ensures continuity in the transverse ties and full anchorage of the ties into the perimeter walls.

It should be noted that car parking at basement level (a definition which may be considered to include service basements and any areas that vehicles are normally permitted to access or manoeuvre, even if no car parking is provided) would automatically require designation of the basement storeys as Class 2B.

12.7.2 Ground floors

The National House Building Council (2010) suggest that ground floor storeys may be excluded from the storey count provided the ground floor columns are designed as Key Elements (page 43). Ground floor columns are by definition those that are often the most vulnerable whether to vehicle impact or explosion loading, the most slender given the frequently larger storey height at ground floor, and with the smallest residual capacity by virtue of being the most heavily loaded. At the same time the ground floor columns are the most critical in terms of the consequences of a collapse following their loss given their support of the greatest number of storeys. While the requirement to designate the ground floor columns as key elements and design them accordingly may follow from the tying and/or alternative loadpath approach adopted in the design of the building, exclusion of the ground floor storey would rarely be anything other than detrimental to the robustness of the structure.

12.7.3 Mezzanine floors

Whether a mezzanine floor should be treated as an additional storey is not defined in either Eurocode 1 or Approved Document A. If the area of a mezzanine is not significant relative to the plan area of the building, it may be argued that the mezzanine does not significantly alter the overall risk of the building to disproportionate collapse and therefore that the mezzanine floor may be disregarded. However, once the plan area of a mezzanine floor is a significant proportion of the area of the building, no such argument can be pronounced, particularly if the mezzanine floor takes structural support from the frame of the overall structure. The question is what constitutes a significant proportion and in what circumstances. To this there is no single definitive answer and determination is made on a case-by-case basis by the Building Control Officer, but as an approximate guide mezzanine floors should be considered as a separate storey if greater than 20% of the building footprint. NHBC suggest the additional limit of 20 m² in addition to the 20% of plan area of the building, the threshold at which a mezzanine or gallery floor should be counted as a storey being whichever is the smaller (National House-Building Council, 2010).

12.7.4 Roof spaces, plant floors and lightweight storeys

Whether roof spaces and rooftop plant floors constitute a separate storey is similarly debatable. Certainly it is clear that an attic space within a domestic dwelling is not a separate storey. If, however, it is converted into habitable accommodation, does it then become a separate storey? The emphasis would in this instance seem to rest on whether the space is habitable. In a commercial office building, if rooftop plant sits directly on the roof slab it would not be included in the storey count. If, however, the plant is enclosed (say for acoustic reasons), does it then become a separate storey through the mere addition of the acoustic enclosure which makes negligible difference to the overall risk of the building to disproportionate collapse? Here again the determination is made on a case-by-case basis, though as a general guide an accessible space should be considered as an additional storey if enclosed by a roof, if the space exceeds approximately 20% of the plan area of the building.

Lightweight storeys and upward extensions on top of existing buildings have been the subject of considerate debate over the years, although the designer should usually adopt the same approach recommended above. When considering the construction of additional storey(s) on an existing building, the Camden ruling (see below) is often cited, which suggests that if any damage within the new top storeys can be contained by the roof slab of the original structure (strengthened if necessary), there is no significant change to the risk to occupants of the original building, and therefore the construction of both the original building and the new storeys may be considered to be Class 2A rather than Class 2B. This approach is typically driven by compromise and is rarely a satisfactory solution.

12.8 Existing buildings

In the design of modifications to existing buildings, the designer must resolve two issues. The first is to determine the extent to which the modification falls under the Building Regulations, and the second is the often significant difficulty of strengthening non-compliant buildings to meet the current robustness requirements. The Institution of Structural Engineers considers in detail the robustness requirements applicable to England and Wales for the design of existing buildings (Institution of Structural Engineers, 2010), discussing in turn extensions, alterations or change of use.

12.8.1 Change of use in the absence of other modifications

Buildings undergoing change of use are subject to Part A of the Building Regulations only in particular circumstances, namely to a hotel, public building (e.g. school/educational establishment, place of worship, library) or institution (e.g. hospital, nursing, residential home or nursery), or from a building that was previously exempt (such as a temporary structure being rendered permanent). In such circumstances are those in which there is an increase in the population risk by virtue of the change in either the type or the extent of the occupancy of the building. The circumstances in which robustness needs to be considered when designing for change of use are therefore relatively limited, although the structural engineer should apply caution in other circumstances which result in a similar increase in risk, for example, the conversion of warehouse or a mill building to blocks of flats. Both in terms of occupancy and structural requirements it might be immaterial whether the warehouse is converted to a hotel or a block of flats, but it would not be a satisfactory solution for robustness requirements to be incorporated in one instance but not the other.

12.8.2 Change of use coupled with extensions resulting in a change of building risk class

Consistent with the above principle of an increase in the population risk brought about by a change of use, it is also broadly accepted that an upward change in the building risk class triggered by the change of use should also necessitate a reappraisal to A3, and the building be brought into compliance with the current requirements.

Extensions are defined as building work by the Building Regulations. Where an extension results in a change in the building risk class, the Regulations require that the structure be designed to comply with the regulations at the time of extension, irrespective of the regulations in force at the time of the original construction of the building. For lateral extensions, it may be possible to construct an argument for the new part of the structure being considered in isolation and the old part of the structure being left unaltered. For upward (e.g. construction of additional floors) or downward extensions (e.g. addition of basement storeys), however, adequate robustness may be difficult to achieve. Nevertheless, the step changes that the requirements given in the Building Regulations and the Approved Document have undergone on two occasions must be acknowledged and the limitations as to the suitability of some types of buildings preceding these step changes to be extended recognised. The first such step change was the introduction of the robustness requirements in the Fifth Amendment in the wake of the Ronan Point collapse, and the second was in 2004 when the requirements were extended to apply to almost all buildings. As such, there may be some circumstances in which the robustness requirements make the proposed change of use or extension untenable, and past determination letters have shown that vertical extensions are likely to be held subject to the current requirements of Approved Document A3 (Secretary of State for Communities and Local Government, 2006, 2008): if the addition of a storey to a Class 2A building raises the classification of the building to Class 2B, either the existing building must be strengthened to meet the Class 2B requirements or the proposed extension must be abandoned. Exceptionally, particular circumstances may justify an alternative approach consistent with Regulation 8, which states that the Building Regulations shall not require anything to be done except '… for the purposes of securing reasonable standards of health and safety': for example, the case may be argued for a loft extension to a four-storey block of residential flats for not assuming the building classification be raised to Class 2B, provided the occupancy of the building does not significantly increase (i.e. the extension will provide additional accommodation for the present occupants, rather than being additional flats). It must be remembered that it is ultimately the responsibility of the Building Control Body to interpret and apply the Building Regulations.

The so-called 'Camden ruling' is sometimes propounded as a suitable approach for upward extensions where the addition of one or more storeys to a Class 2A building raises its classification to Class 2B. The Camden ruling, formalised in the mid-1980s by Camden Building Control, suggests that if the design incorporates a strong floor at the original roof level designed (including its supporting structure) to be capable of taking the debris load from the storeys above, the building may be classified as Class 2A rather than Class 2B, meaning horizontal and vertical ties are not required. Consideration of dynamic effects was required, with a dynamic load factor stated as 3.0 for roof loadings and 2.0 for walls (Heyne, 2006). This approach has been accepted in the past by Building Control Bodies as demonstrating a reasonable level of robustness, should the new construction collapse, to prevent further collapse through the entire building which would be considered disproportionate. However, while DCLG do not comment on matters unless prescribed by the Approved Document, the Camden ruling is not favoured by DCLG (Carpenter, 2007) and it has generally now fallen out of favour amongst Building Control Officers. As such, in considering whether to base a design on the Camden ruling, the structural engineer must consider whether he or she would still argue in the aftermath of a collapse affecting the original part of a building following new construction of additional storeys above that the risks to occupants of a building were no worse than before the extension, and hence that compliance with Class 2B requirements was unreasonable. Whether a risk is significantly greater depends on the perspective of the observer: had the extension not been built, there would have been nothing to bring about the collapse of the building.

12.8.3 Extensions resulting in no change of building risk class

Where there is no change in the building risk class as a result of the extension, the extension is usually considered under Regulation 4(3), which states that building work shall be carried out so that after the work is completed, the building complies with the robustness requirements, or, where it previously did not comply with the requirements, is no more unsatisfactory than it was before. The extension must comply with the robustness requirements regardless of the condition of the existing building. This principle that a building is made no more unsatisfactory than it was before the extension is an important one, which should be judged both in terms of the structural considerations and the occupancy of the building. The diligent structural engineer should, however, seek to apply regulation A3 as far as reasonably practicable: where measures can be straightforwardly achieved that improve the structural integrity of the building as part of the renovation at relatively little cost, for example, incorporation of anchorage details between slabs and load-bearing walls if the façade of the building must be demolished for construction access and subsequently rebuilt, it would be remiss of the engineer not to do so.

12.8.4 Material alterations

The final category of building work on an existing building that might trigger consideration of the robustness requirements is a material alteration. An alteration is *material* if a building that previously complied with the robustness requirements would no longer comply, or if a building that previously did not comply with the requirements were made more unsatisfactory. In both cases, consideration is given to the state of the building at any stage of the building work, rather than merely the final condition of the building. Alterations may or may not incorporate an extension, discussed above. Alterations not incorporating an extension which might adversely affect the robustness of the building include, for example, the incorporation of a lift core into a block of flats to provide level access, the breaking out of the first-floor slab to create a double-height lobby or the construction of an atrium in the building.

Structural alterations without extension may or may not be accompanied by a change of use which results in an upward change in the building risk class. A building whose class increases through the change is likely to be made less satisfactory in relation to Approved Document A than it was before, because the change in risk class brings with it a requirement for a higher level of robustness measures for the building (National House Building Council, 2010).

12.8.5 Requirements for existing buildings in Scotland

The requirements in Scotland exceed those in England and Wales, requiring all buildings undergoing *conversion* to be altered or strengthened to the standard required by current regulations so far as reasonably practicable, and in no case be worse than before the conversion. A *conversion* is a wide-ranging definition that in several areas exceeds the requirements for buildings undergoing change of use applicable in England and Wales, specifically:

- Changes in the occupation or use of a building to create a dwelling or dwellings.
- Changes in the occupation or use of a building which alters the number of dwellings in the building.
- Changes in the occupation or use of a building so that it becomes a residential building.

In Scotland, extensions and alterations are treated as *construction* in the same way as new construction. Unlike in England and Wales, there are no qualifications: the Technical Handbooks apply in their entirety (Scottish Building Standards Agency, 2010a, 2010b). Alterations refer to work carried out on an existing building where no change of occupation or use is involved. In addition to the full current standards relevant to alteration work, as in England and Wales the whole building must not, as a result of the alteration, fail to comply with building regulations if it complied originally, or fail to a greater degree if it failed to comply originally. Alterations to an existing building that are part of a conversion are subject to a wider application of the regulations, so that the building being converted complies more fully as described above (Historic Scotland *et al*., 2007). The requirements for extensions are similar to those in England and Wales.

12.9 Methods for design for structural robustness

Current methods for design for structural robustness are threefold: tie force methods, alternative loadpath methods and key element design, briefly described in the paragraphs below. In the UK context, tie force methods and key element design are prescriptive methods while the alternative loadpath approach is a quantitative approach. Any of these methods may be employed within a risk-based framework, which is also described.

12.9.1 Tie force methods

Tie force methods are based on the development of a modest degree of additional robustness in structures that provides additional structural integrity and ability to redistribute load following local damage. To be effective, ties must be designed and detailed as follows.

12.9.1.1 Horizontal ties

Horizontal ties should be designed and detailed as follows.

- Ties should be provided in two approximately orthogonal directions and should be continuous in both directions in the horizontal plane through the structure.
- Perimeter columns should be anchored with transverse horizontal ties made continuous with the internal ties.
- Edge ties should be made continuous around the perimeter of the building.
- The tying system should be designed so that no column is tied in only one horizontal direction.
- Wherever practicable ties should be distributed throughout each floor and roof level.
- If beams are used as ties, floor slabs should be effectively anchored in the direction of their span either to each other continuous over their supports or directly to their supports.
- Direct and robust connection should be provided between the horizontal ties and vertical elements.
- At re-entrant corners or at substantial changes in construction, care should be taken to ensure that the ties are adequately anchored or otherwise made effective.
- Where the building is divided by expansion joints into structurally independent sections, each section should have an appropriate and independent tying system.

See **Figure 12.3** for an example of general horizontal tying of a steel-framed structure.

Approved Document A permits 'effective anchorage of slabs to walls' in Class 2A buildings as a variant of horizontal ties. This was introduced in 2004 and is an apparent relaxation of the requirements relevant to timber and load-bearing masonry construction. The phrase refers to standard joist hanger details and similar in BS 5268-2 and BS 5628-1 (BSI, 2002a, 2005a) developed to provide simple lateral restraint to movement of load-bearing walls rather than prevention of disproportionate collapse. The structural engineer should always aim to ensure that ties in a Class 2A building incorporate the principles outlined above and are continuous through the structure (not stipulated for effective anchorage), and that the robustness of the tie details is comparable to the horizontal ties required for a building designated as Class 2B.

12.9.1.2 Vertical ties

Vertical ties, where provided, should be designed and detailed as follows.

- Each column and each wall carrying vertical load should be tied continuously from the lowest to the highest level.
- Columns should be made continuous through each beam–column connection.
- Columns should be designed for a tensile force equal to the largest total ultimate vertical dead and imposed load applied to the column at a single floor level (i.e. the sum of the reactions from all the beams/slabs connected to a column at that floor level) to which the column may be subjected in tension if support is lost from below.
- Vertical ties should be robustly connected to the horizontal tie system.

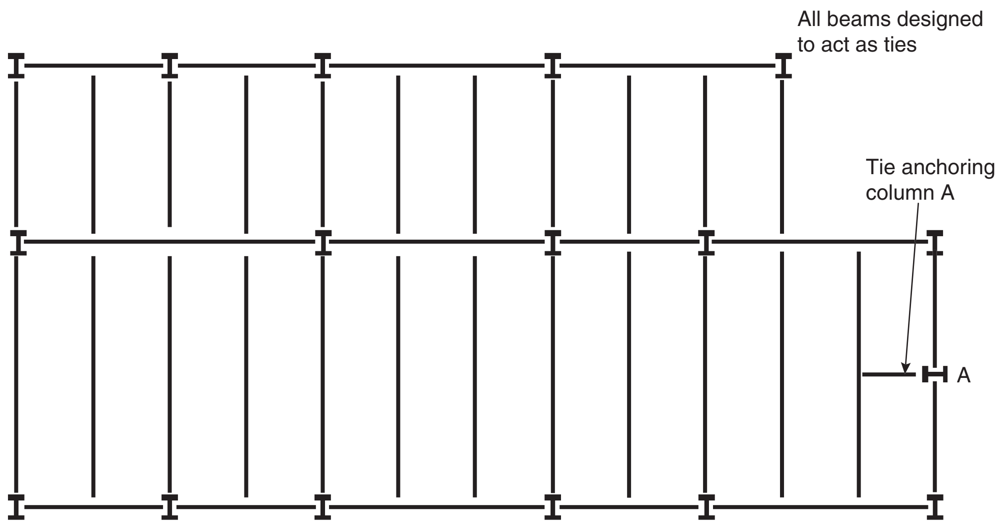

Figure 12.3 Example of general tying of a framed building (BSI, 2001). Permission to reproduce extracts from BS EN 1991-1-7:2006 is granted by BSI

12.9.1.3 Design principles

The intention of providing vertical ties in addition to horizontal ties is to distribute the loads between all the floors, recognising that some floors will be less heavily loaded as others and therefore that additional capacity will be achieved if all the floors are mobilised together. As such, vertical ties provide an additional level of robustness over that achieved by horizontal ties alone, which will assist the remaining structure to redistribute the loads after an accidental event.

Tying systems are not meant to fully describe the structural mechanics of arresting a collapse following localised damage. Instead ties are prescriptive rules for low- to medium-risk structures which are intended to produce structures that exhibit a level of robustness which may be considered sufficient in accidental circumstances, thereby reducing the risk of disproportionate collapse to a tolerably low level. A key limitation of tie-force methods is that with their usually prescriptive nature, it is not usually possible to quantify the level of robustness achieved or to demonstrate adequate robustness against a particular localised damage scenario such as the notional removal of any given column, but merely compliance with Code requirements.

Tying systems will not necessarily be in themselves sufficient to fully arrest a collapse following a given localised damage scenario, but comparison of buildings designed pre- and post-Ronan Point in past vehicle-borne explosive attacks in Manchester and London during the Irish Republican terrorism campaign in the 1990s has demonstrated the positive effect of tying in limiting both the extent and severity of the structural collapse (Moore, 2002; Sadek, 2008; Cormie *et al*., 2009).

Well designed and detailed ties will result in an enhanced degree of continuity, ductility, and ability to transfer load to other parts of a structure such that the overall robustness of the structure is enhanced. However, unlike in seismic design, there are no requirements for ties to be designed to assure ductility; however, research has shown that the absence of ductility can have a significant reduction on the level of robustness achieved (e.g. Merola *et al*., 2009). The structural engineer should therefore consider issues such as the distribution of ties within the slab which are known to affect ductility and consequently robustness.

12.9.2 Alternative loadpath methods

Alternative loadpath methods are a quantitative approach based on engineering analysis and design to demonstrate sufficient robustness to withstand a given localised damage scenario. Usually this scenario is the notional removal of a single load-bearing element (whether column, wall or beam) with the required performance criterion being the demonstration that the resulting collapse is limited to not more than 15% or 100 m^2 of the area affected floor, not extending further than the immediately adjacent storeys (BSI, 2006). BS 5950-1:2000 is more stringent, limiting the portion at risk of collapse to the affected floor and one immediately adjoining floor level, either above or below (BSI, 2000).

When a column is lost from the structure, the gravitational load (dead + live load) is applied to the beams that connect into it, which act as an alternative loadpath in transferring this load to the adjacent columns. If the elements that form this loadpath are capable of withstanding this load in addition to their existing loads, the collapse is arrested and the structure is stable in its damaged state (**Figure 12.4**). If, however, these elements do not have sufficient residual capacity to withstand the additional demand, they also fail and the collapse propagates. A similar process follows until (and if such point is found) the structure offers sufficient residual capacity to arrest the collapse.

Five mechanisms are fundamental to the robustness problem (Cormie, 2009) and are illustrated in **Figure 12.5**, namely (a) catenary action in the structural frame, (b) shear deformation of transfer structures, (c) membrane action in structural

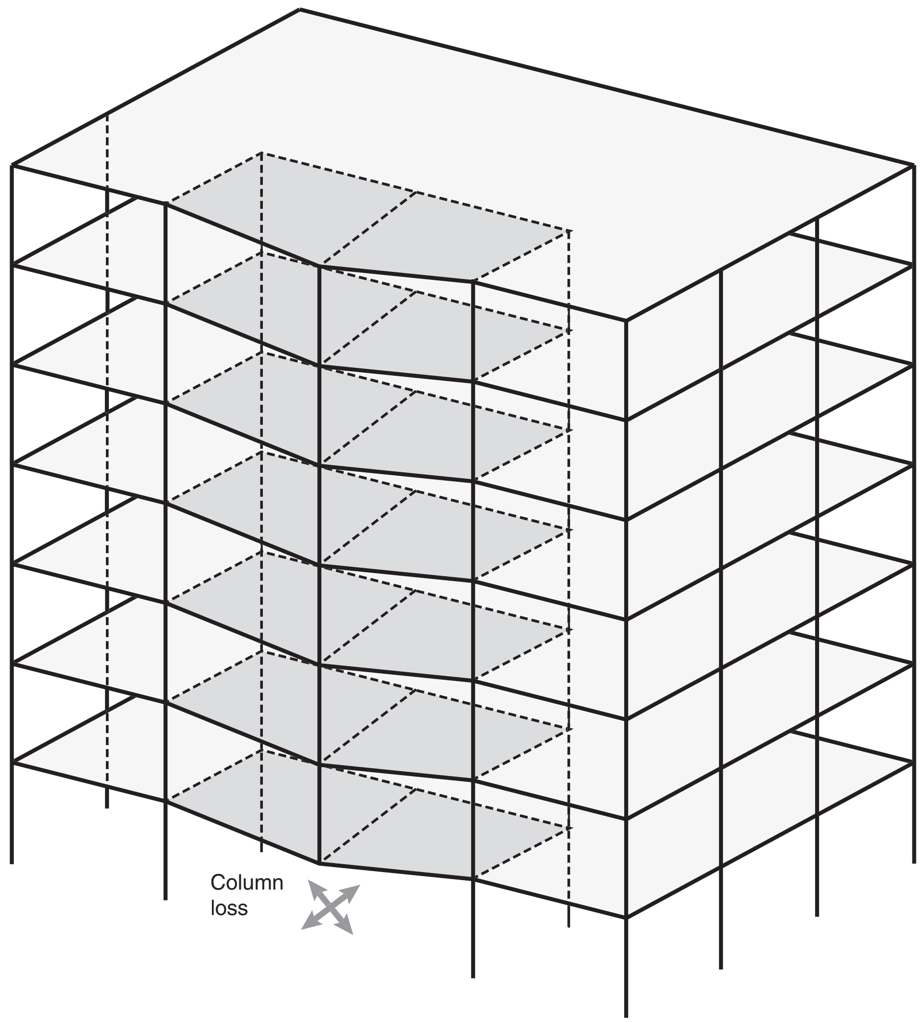

Figure 12.4 Sudden column loss. Adapted from Izzudin *et al.*, 2007

slabs, (d) Vierendeel action, and (e) compressive arching in the beams and/or floor slabs. For most structures, the redistribution of load solely through the classical mechanism of catenary action shown in (a) is not possible and successful redistribution of load through alternative loadpaths relies on the successful mobilisation of one or more of the mechanisms shown in (b) to (e). In some types of structure, it may also be possible to develop compressive strut action in masonry (f) or similar, the load-bearing capacity of which can be significant.

12.9.2.1 Arrest of collapse

When undertaking an alternative loadpath analysis, the dynamic effects of the load should be considered. Approved Document A describes the removal of the column, wall or load-bearing beam as 'notional', but consideration of the forces involved solely as static forces will underestimate the problem. Where structures are designed and detailed so that they can develop significant ductility post-yield, a dynamic load factor in the range 1.3 to 1.5 may be justified; however, for structures that have little ductile capacity and must therefore be designed to remain broadly elastic, a dynamic load factor of 2.0 is necessary.

12.9.2.2 Accidental loadcase

Eurocode 0 (BS EN1990) defines two loadcases for a typical office building (BSI, 2002b) as follows, the partial load factors taken from the UK National Annex (BSI, 2005b) and applicable in the UK:

$$1.0\ G_k + (0.5 \text{ or } 0.0)\ Q_k + 0.0\ W_k + 1.0\ A_k \qquad (12.1)$$

$$1.0\ G_k + (0.3 \text{ or } 0.0)\ Q_k + 0.2\ W_k + 1.0\ A_k$$

where

G_k = dead load

Q_k = imposed load (partial factor depends on whether action is adverse or beneficial)

W_k = imposed load

A_k = accidental load.

In buildings predominantly used for storage or where the imposed load is otherwise of a permanent nature, the full imposed load Q_k should be used.

The above compares with accidental loadcases in the British Standards typically defined:

$$1.05(1.0\ G_k + (0.33 \text{ or } 0.0)\ Q_k + 0.33\ W_k + 1.0\ A_k) \quad (12.2)$$

The accidental load A_k in a column removal scenario is the load that was carried by the column prior to its removal under an accidental loadcase, multiplied by the relevant dynamic load factor described above. This is the load that must be transferred through alternative loadpaths if the structure is to remain stable.

It is good practice to design the structural slab for the debris load associated with the area of collapse of the slab(s) above, in order to ensure successive floor collapse does not occur. If simply supported, the full mass of the structural slab and any supported finishes should be applied assuming a dynamic load factor of 3.0 and a partial factor of 1.05 for accidental dead load. If the floor slab is of continuous construction, it is reasonable to assume that only a proportion of the floor slab will collapse. The extent is for the engineer to determine on an individual basis, but if 50% of the weight of the floor is applied with a dynamic load factor of 3.0 and a partial factor of 1.05, the loadcase will often be less onerous than the loadcase for the design of the floor slab at the ultimate limit state.

In load-bearing wall construction, the Approved Document requires the removal of a length of load-bearing wall equal to 2.25 times the storey height H. Where columns are close-centred there is no such requirement; however, it is good practice to

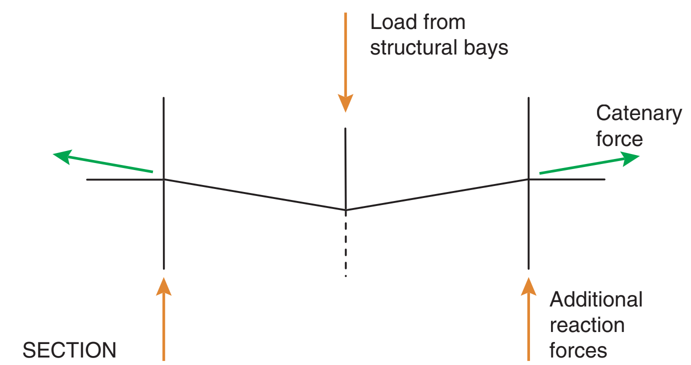

Figure 12.5(a) Mechanisms to resist collapse. Catenary action in structural beam/column frame of an internal column after removal of a supporting column. © Arup

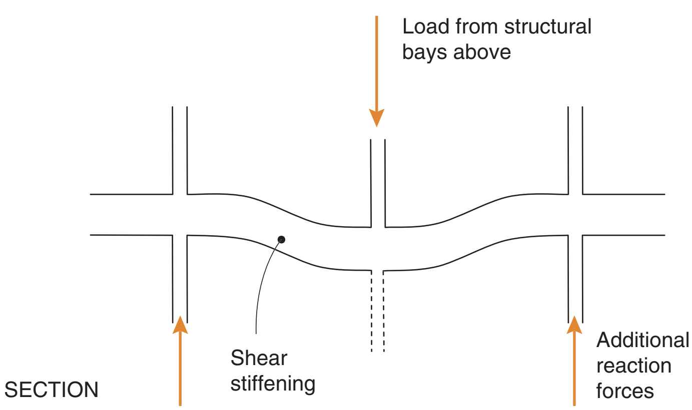

Figure 12.5(b) Mechanisms to resist collapse. Shear deformation of deep transfer/spandrel beams. © Arup

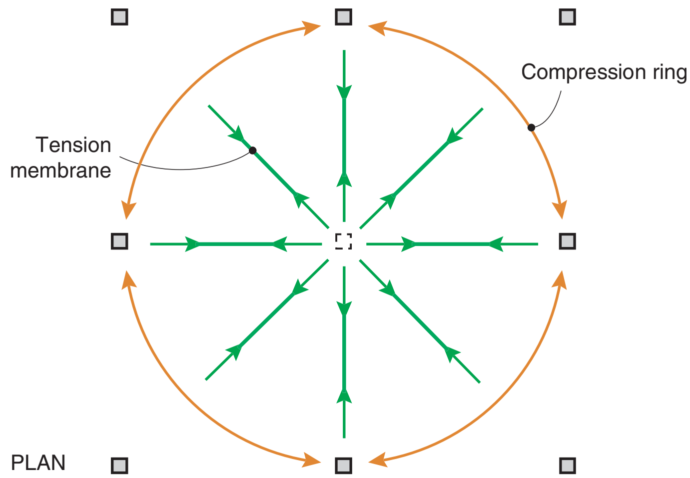

Figure 12.5(c) Mechanisms to resist collapse. Tensile membrane developed in a flat slab after the removal of the central column. © Arup

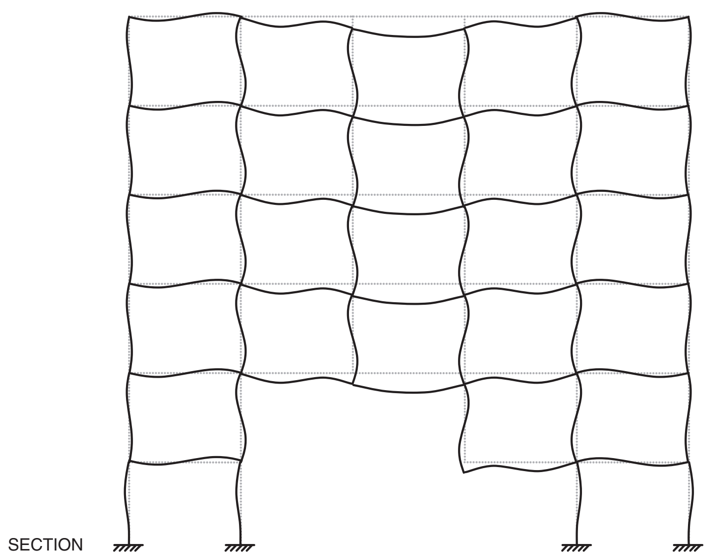

Figure 12.5(d) Mechanisms to resist collapse. Vierendeel action due to moment capacity in beam/column connections following loss of two columns (of which one is lost over two storeys) and the first floor beam over two structural bays. © Arup

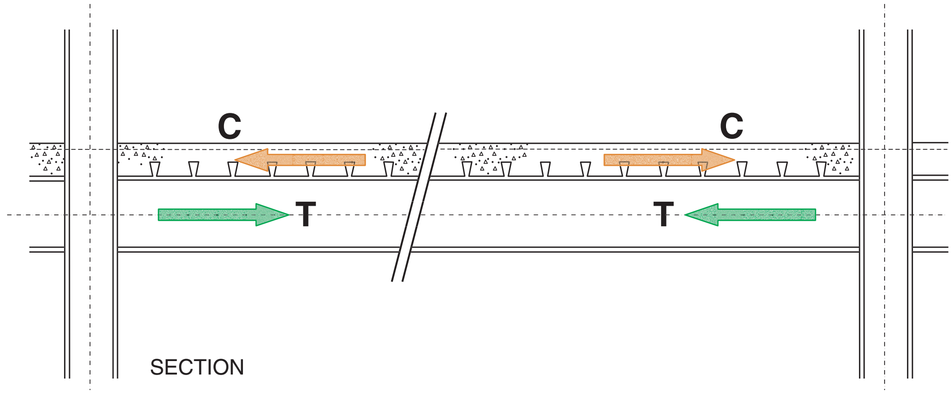

Figure 12.5(e) Mechanisms to resist collapse. Compressive arching action between composite metaldeck slab and steel floor beams. © Arup

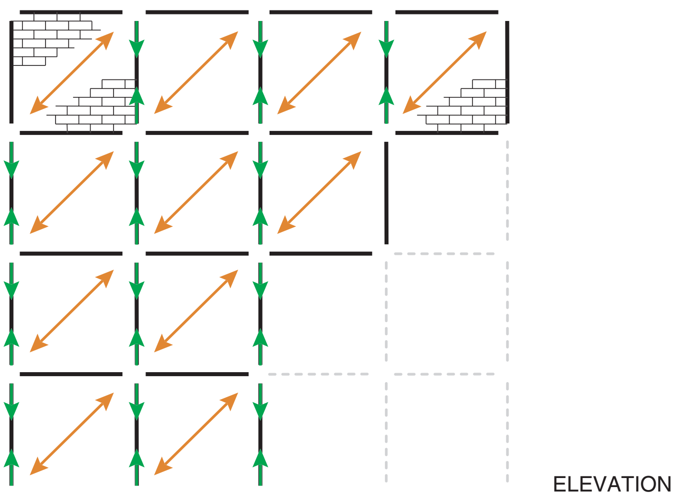

Figure 12.5(f) Mechanisms to resist collapse. Compressive strut action in masonry panels © Arup

consider multiple columns to be removed over the same length of 2.25H. Similarly, for inclined columns supported from a common node, the engineer should consider circumstances which could give rise to the failure of the node and therefore the loss of load-bearing capacity in multiple columns.

12.9.2.3 Analysis methodology

Successful alternative loadpath analysis generally requires a nonlinear analysis technique. If linear analysis is used, the structure must be designed to remain broadly elastic; however, design of the structure to remain within its elastic limit under the accidental loadcase will usually lead to an overly conservative design. When 'overstressing' in an elastic analysis reaches a moderate level, the results will become invalidated and a nonlinear approach must be used incorporating material nonlinearity. Geometric nonlinear effects are also usually important in a collapse analysis and should normally be included.

Material ductility may be modelled in a number of ways, the most simple being a linear elastic-perfectly plastic material model as shown in **Figure 12.6**. In some cases, it may be appropriate to incorporate strain hardening, **Figure 12.7**. Strain rate effects may sometimes be considered: at high strain rates, many materials exhibit enhanced yield strength, although it will normally be conservative to ignore it.

Whatever the material model assumed for the parent material, connections will require particular consideration. Connections are typically stiffer and therefore less ductile than the parent material, but are the locations at which yielding will be concentrated and therefore the locations at which failure is most likely to occur. While careful detailing will maximise the inherent ductility of the connections, most simple connections typical of UK construction are relatively brittle with a limited rotational ductility capacity, particularly when rotation and axial load are combined as is required for catenary action. Consequently, it is highly likely that the design of the connections will significantly influence the ability of the structural system to redistribute load.

Geometric nonlinearity may be significant in an alternative loadpath analysis: the large displacements necessary to develop some of the mechanisms of resistance illustrated in **Figure 12.5** (a) to (f) usually being sufficient to invalidate the small-displacement theoretical assumptions typically adopted in normal structural design. Ignoring large-displacement effects may be non-conservative and result in an unsafe design.

The ultimate capacity of the structural system often involves softening and subsequent hardening of the response as compressive arching is overcome and catenary modes develop. A displacement-controlled algorithm is required for such solutions: force-controlled algorithms found in many structural analysis packages are unable to calculate the response beyond the point where the stiffness first becomes negative, which typically neglects a significant proportion of the strain energy capacity of the structure. The most accurate solution will be obtained using a nonlinear dynamic time history analysis; however, this is also the most complex and solutions giving a reasonable approximation can usually be obtained using nonlinear static methods, particularly if displacement-controlled.

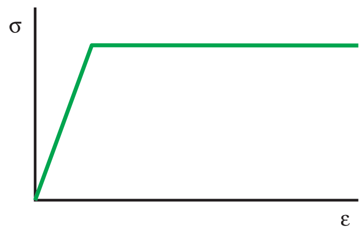

Figure 12.6 Linear elastic-perfectly plastic material model

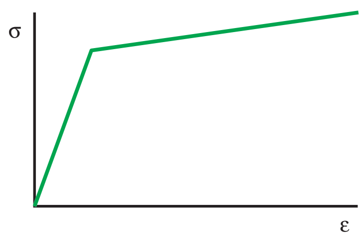

Figure 12.7 Bi-linear elasto-plastic material model with strain hardening

The dynamic effects may be taken into account utilising a dynamic load factor as described above, or alternatively may be calculated directly through energy balance between the strain energy capacity of the structure and the work done by the applied load, expressed dynamically (**Figure 12.8**).

Performance criteria for the limiting ductilities to which members and connections may be subjected are found for seismic design in ASCE 41 (American Society of Civil Engineers, 2006), which have been adapted for progressive collapse analysis in UFC 4–023–03 (United States Department of Defense, 2010) and may be adopted in an alternative loadpath analysis. Data for simple connections in steel construction, however, are limited, and performance criteria which account for the contribution of the structural slab are not generally available.

12.9.3 Key element design

Approved Document A states that where the notional removal of columns/lengths of load-bearing walls would result in a collapse that exceeds the limits on the tolerable area at risk of collapse (i.e. where horizontal and vertical tying has been implemented in a building whose structural grid is large, or where it has not been possible to show sufficient resilience in an alternative loadpath analysis), the element should be designed as a 'key element'. Key elements are designed for enhanced loads to provide an additional level of robustness and decrease the likelihood of failure under a range of accidental loads to which the element might reasonably be subjected. Inasmuch as the protection of the element has been shown to be critical to ensuring a disproportionate collapse does not occur, Approved Document A recommends key element design only as the method of last resort if horizontal and vertical tying or alternative loadpath analysis alone is insufficient.

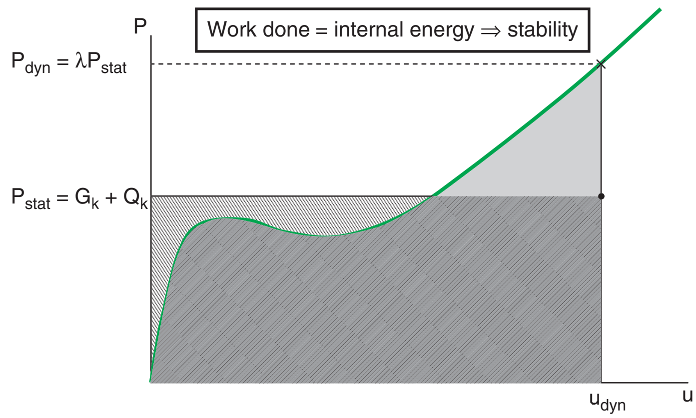

Figure 12.8 Work done versus internal energy © Arup

Key elements should be designed in accordance with the following guidance:

- The element should be designed for an accidental loading of 34 kN/m² applied to the width of the element and any supported cladding.
- Any element that provides lateral restraint vital to the stability of a key element should itself also be designed as a key element for the same accidental loading.
- The accidental loading should be applied to the member from all horizontal and vertical directions, in one direction at a time, together with the reactions from other building components attached to the member subject to the same accidental loading, but limited to the maximum reactions that could reasonably be transmitted considering the breaking resistances of such components and their connections.
- The applicable accidental loadcase assumed should be that given above (equation 12.1), except in buildings predominantly used for storage or where the imposed load is otherwise of a permanent nature. In such cases the full imposed load should be used.
- The imposed accidental load of 34 kN/m² should be applied in combination with the dead load using a partial load factor $\Psi = 1.0$ together with (typically) $\Psi = 0.5$ for the imposed load and $\Psi = 0.2$ for the wind load.

In adopting a key element approach, there are two essential aspects to which the structural engineer must give consideration in the design. The first is that the rules for key element design were first developed in the wake of the Ronan Point collapse when structural grids were much smaller than in modern practice. In the late 1960s a 6 × 6 m grid was typical, the limit of 70 m² given in the Fifth Amendment corresponding to two such perimeter bays. In Eurocode 1 this limit is increased to 100 m²: this does not reflect a greater tolerability of risk but is merely a necessity reflective of increasing structural spans, corresponding to two perimeter bays on a 7.5 × 7.5 m grid. However, even with this increase the limiting area of collapse is frequently exceeded purely by virtue of the grid size, which in commercial office construction can frequently be anything up to 13.5 × 18 m. As such, the designation of elements as key has become commonplace and almost the norm, whereas the principles were conceived with the assumption that key elements would be the exception.

The second aspect the designer must consider follows from this, namely that with the much more commonplace nature of key elements the designer must consider whether a load of 34 kPa is appropriate for elements that have been shown to be critical. 34 kPa is equal to 5 psi, a rounded estimate of the explosion pressure estimated to have caused failure of the precast concrete load-bearing flank wall panel at Ronan Point, based on observational and experimental evidence (Moore, 2002). Consequently it was recommended as a suitable design pressure for load-bearing wall panels in large-panel structures for which it could not be shown that the loss of the panel could be sustained without resulting in a disproportionate collapse. The load has remained enshrined in Codes of Practice ever since. It is unfortunate that the numeric value in metric units suggests a degree of precision which is unintended and undeserved. It should be noted that previous versions of Approved Document A referred to a design load of 'at least' 34 kPa applied from any direction (Department of the Environment and The Welsh Office, 1985).

When conceived, 34 kPa was a relatively onerous load in most circumstances, certainly leading to an enhancement of the element design over that required by other loadcases. However, in modern construction columns are heavier due to the greater loads resulting from the increased grid size. The effect is further exacerbated in high-rise construction so that the load is often a trivial loadcase that is bounded by other loadcases. In addition, cladding typically spans floor-to-floor rather than loading the column, decreasing the loaded width for consideration in key element design and further decreasing the impact of the requirement. The structural engineer should give careful consideration to the selection of a design load for key elements which is appropriate to the critical nature of the elements. The design load should normally be such that it results in an enhancement of the element design, unless through a risk assessment it can be shown that no reasonably foreseeable or unforeseeable hazards will result in the failure of the element.

12.10 Systematic risk assessment for design of Class 3 buildings

12.10.1 Basis of a systematic risk assessment

For Class 3 buildings, Approved Document A recommends that 'a systematic risk assessment of the building should be undertaken taking into account all the normal hazards that may reasonably foreseen, together with any abnormal hazards'. Eurocode 1 goes further, calling for the systematic risk assessment to take into account 'both foreseeable and unforeseeable hazards'. While it is difficult to assess and design for unforeseeable hazards by their very nature, the concept of the Eurocodes is that strategies based on the control of risk from identifiable hazards should be implemented, in parallel adopting strategies based on limiting the extent of localised failure through a minimum level of inherent robustness irrespective of whether there are any hazards the designer can foresee. As a minimum, a Class 3 building should be generally expected to satisfy the Class 2B robustness requirements, sometimes also incorporating additional mitigation measures which the risk assessment finds to be necessary.

The design approach for Class 3 buildings therefore requires a fundamental consideration of the hazards to which the structure might reasonably be subjected, and an assessment of the risks to building occupants (and others who might be affected by damage to the building, for example, the general public in the vicinity of the building and the occupants of neighbouring buildings or in close proximity) based on the likelihood and the consequences associated with each hazard. The risk assessment should not necessarily be limited to consideration solely of accidental hazards: in some cases, malicious actions are a foreseeable hazard. Two such examples are safety-critical vandalism and the blast effects of a detonation due to explosive terrorist attack. Where relevant such malicious hazards must therefore be considered. (Consideration of terrorist actions should normally be undertaken by a suitably qualified member of the ICE Register of Security Engineers and Specialists, www.ice.org.uk/rses.)

As discussed in Chapter 3: *Managing risk in structural engineering*, risk may be defined as:

Risk	=	Likelihood	×	Consequence
(associated with a particular hazard)		(or probability)		(or severity)

Each building differs in terms of its sensitivity to accidental hazards and the consequences of failure. A systematic risk assessment for design of a Class 3 building should seek to eliminate the risk of disproportionate collapse so far as reasonably practicable. This is derived from the designer's legal duty under the Health and Safety at Work Act (1974) to reduce risk within the scope of their undertaking so far as is reasonably practicable. The requirement to exercise a legal duty so far as is reasonably practicable is at the heart of health and safety legislation in UK law, and acknowledges that it will be impracticable to eliminate all risk associated with foreseeable (and unforeseeable) hazards. Thus the duty is not absolute, but considers the progression in cost/benefit terms from proportionate through to disproportionate action, implying that measures are required up to a point of disproportion, but not up to the point of gross disproportion.

The control of a given hazard is intended to be proportionate to the risk posed. For further discussion of the management of risk in structural engineering, refer to Chapter 3: *Managing risk in structural engineering*.

12.10.2 Hazards

Some of the hazards that should be considered in a Class 3 risk assessment are listed in **Table 12.2**. The list is illustrative only and is not exhaustive. Indeed, it is strongly preferable for the designer to give fundamental consideration to the hazards that might occur in each circumstance, rather than to use a checklist or prescribed list of hazards.

The basic assumption underlying a Class 3 systematic risk assessment is that the buildings categorised as Class 3 are such either because the hazard consequences (usually in terms of potential loss of life but sometimes also safety or financial loss) or the likelihood of one or more hazards occurring are greater than in Class 2. The classification of the building as Class 3 is itself a decision that results from a high-level risk assessment on this premise, and therefore the systematic risk assessment may be focused on the physical effects of the hazard in terms of the extent of the structure at risk of collapse.

A key difficulty in a systematic risk assessment is the treatment of low likelihood/high consequence events for which a quantitative assessment is often meaningless. However, for such events the consequences are often so onerous that should the hazard materialise it will not be deemed a tolerable event. Here, therefore, the designer should consider whether it is foreseeable that the hazard will materialise, and if so focus on determining measures through which the risk can be mitigated so far as reasonably practicable. In doing so, the risk assessment for such hazards effectively becomes a conditional sum used to determine mitigation measures necessary, given that there is a finite likelihood that hazard could materialise and the consequences would automatically be disproportionate.

12.10.3 Uncertainty

The risk assessment should also consider the uncertainty of the assumptions made. This uncertainty can substantially affect the determination of the risk, a sensitivity which increases exponentially as likelihood and consequence move into the 'tail' of the distribution. As such, it is necessary to undertake a sensitivity analysis as part of the risk assessment, particularly for low likelihood/high consequence events where the uncertainty and sensitivity are likely to be greatest. The sensitivity of the risk to the underlying assumptions can result in the need for more extensive mitigation over that suggested by the 'central' values of the likelihood and consequence. For low likelihood/high consequence events, a 'cliff-edge' analysis may sometimes be warranted, the purpose of such an analysis being to demonstrate that there is a gradual change in the structural behaviour beyond the design value. Where a so-called cliff edge is identified, measures should be implemented either to change the structural behaviour so that the cliff edge is removed, or to push the cliff edge to a point sufficiently above the design value that the risk associated with the uncertainty of the assumptions is mitigated.

Design and construction
■ Calculation error
■ Robustness during construction (or demolition/alteration)
■ Sub-standard construction
■ Gross construction error
■ Material defects
■ Sub-standard components, for example, due to counterfeiting of Quality Control markings/certification
■ Dropped object
■ Unauthorised alteration
Permanent, imposed and environmental actions
■ Wind, snow, rainwater ponding, flooding
■ Excessive floor loading
■ Earthquake
■ Fire
■ Structural deformation/movement
■ Subsidence/ground movement
■ Groundwater level change
■ Undermining of foundations
■ Fatigue
■ Corrosion/rot
Accidental actions
■ Vehicle impact
■ Gas explosion
Malicious actions
■ Safety-critical vandalism
■ Explosive terrorist attack
Combined hazards*

* A combination of hazards is not necessarily as straightforward as multiplication of the two independent risks. After some events a second hazard can become highly likely, for example, vehicle impact or sudden ground movement leading to flooding or to a gas release and potential gas explosion or fire. Equally the consequences of a second otherwise independent hazard may increase, for example, where fire protection is lost in a vehicle impact such that structural collapse could occur if a fire does happen. The World Trade Center exhibited remarkable robustness under the aircraft impact with the loss of several columns, but collapse was eventually due to the loss of fire protection in the initial impact and the ensuing fire.

Table 12.2 Examples of hazards

12.10.4 Determination of risk

The central part of a systematic risk assessment is the determination of the risk for each hazard through consideration of the two constituent parts, likelihood and consequence. This is followed by consideration of what is necessary to mitigate the hazard so far as reasonably practicable as described above.

The risk assessment may take any form appropriate to the particular case under consideration. This may be qualitative, semi-quantitative or in some cases fully quantified (**Table 12.3**).

Qualitative
■ Likelihood classed as 'unlikely', 'likely', etc.
■ Consequences classed as 'minor', 'severe', etc.
■ Risk determined from a qualitative combination of likelihood and consequence, sometimes with a risk matrix (e.g. Figure 12.9)
Semi-quantitative
■ Likelihood prescribed values on say a five-point scale based on description of how likely it is that the hazard will occur (e.g. once per year, once during the lifetime of the building)
■ Consequence prescribed values on a similar scale, often based on descriptions of the potential fatalities/injuries (e.g. few minor injuries, several minor injuries/few major injuries, one fatality, several fatalities), financial loss, extent of collapse or some other metric
■ Risk determined from a risk matrix combining the likelihood and consequence (e.g. Figure 12.9)
Quantified risk assessment
■ Probability of the hazard occurring calculated and expressed quantitatively (e.g. 1E–4/yr)
■ Consequences calculated, usually based on fatalities and injuries expressed as financial 'cost'
■ Individual and population risk calculated for each hazard and aggregated across all hazards

Table 12.3 Types of risk assessment

Common to all forms of risk assessment is the concept of a tolerable threshold, beyond which mitigation must be applied to bring the residual risk back within tolerable bounds. The bold line in **Figure 12.9** shows an example of this threshold, but it must be noted that no one likelihood and consequence scale will be universally applicable, and similarly the risk appetite must be discussed and agreed with the client and building control authority at the outset. The example in **Figure 12.9** shows a relatively risk-averse scale skewed against hazards resulting in major consequences, reflecting an aversion to such hazards which is often observed in society when such hazards do materialise. This aversion is observed despite the risk associated with the hazard being lower than for some more frequent hazards when assessed using the conventional type of risk matrix discussed in Chapter 3: *Managing risk in structural engineering* – in effect, the perception of risk becomes one of conditional probability: should the hazard materialise, it will automatically be perceived as disproportionate. As such, the societal perception of risk is perhaps irrational: we do not expect buildings to collapse, and when they do it is rarely viewed favourably by society. The structural engineer must therefore be careful to avoid the 'one size fits all' risk matrix and design a matrix which is suited to the particular circumstances of the project and incorporates the client's and building control authority's attitude to risk. It is the responsibility of the structural engineer to ensure that the client's decision on tolerability of risk is an informed one, based both on the engineering consequences of a particular hazard and the legal, societal and other implications of the client's decisions as the risk owner.

Fully quantified risk assessments are usually only used in particular circumstances, typically in the nuclear, petrochemical and other low likelihood/high consequence industries. In a semi-quantitative risk assessment that would be typical for a Class 3 risk assessment, the values ascribed to the likelihood and consequence of the hazard would be determined by the designer in a systematic fashion. (See **Tables 12.4** and **12.5** for an example of each.)

Once the likelihood and consequence are defined, the risk can be determined from the risk matrix. Any risk assessment process requires the threshold between tolerable and intolerable risks to be identified: this is represented as an example by the dark line in **Figure 12.9** but needs to be agreed with the client and control authority at the outset of the risk assessment.

The principle that applies to mitigation of the risks identified is that intolerable risks must be mitigated, but more widely *all* risks should be mitigated so far as reasonably practicable. The *ERIC* (Eliminate, Reduce, Isolate, Control) hierarchy defining an approach to the reduction of risk (Institution of Structural Engineers, 2012) is particularly suitable for engineering

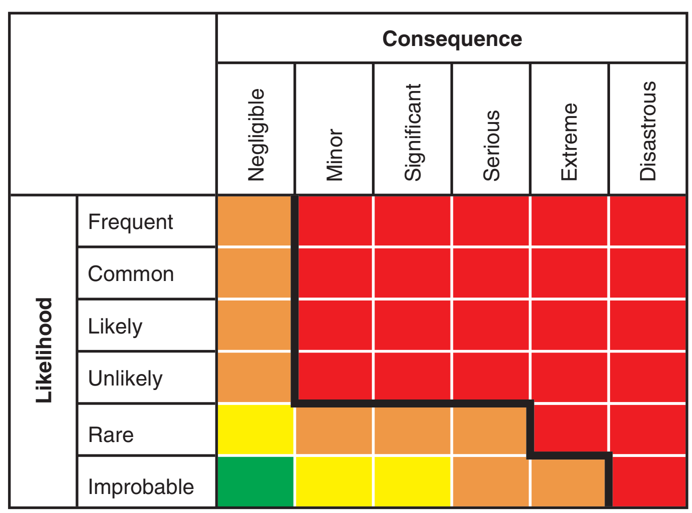

Figure 12.9 Example risk matrix adapted from Harding *et al.*, 2009. Courtesy of *The Structural Engineer*

Likelihood of event	Frequency
Frequent	More than 10 per year
Likely	Between 1 and 10 per year
Occasional	Once every 1 to 10 years
Unlikely	Once every 10 to 100 years
Rare	Once every 100 to 1000 years
Improbable	Less than once every 1000 years

Table 12.4 Likelihood categories

Likelihood of event	Frequency
Disastrous	More than 20% collapse of building
Extreme	15% collapse of floor or 100 m² to 20% collapse of building
Serious	Lesser of 15% collapse of floor or 100 m²
Significant	Loss of structural member local to hazardous event but no collapse of floor
Minor	Local structural damage but no loss of structural members
Negligible	Superficial damage only

Table 12.5 Consequence categories

systems. Most preferable is that the risk is *eliminated* in the first place, by removing the hazard that is the cause of the risk. If this is not possible, ways should be sought of *reducing* the risk associated with the hazard by replacing the cause of the hazard with something less dangerous. The third option is to *isolate* the hazard by providing a means of protecting against it, and finally to *control* the hazard by exposing people to less of the hazard (or fewer people to the hazard), or mitigating (reducing) the consequences of the hazard if the risk occurs. Design of a structural column to withstand an under vehicle impact falls into the last of these options, *controlling* the hazard should the risk materialise. However, it is far better to *eliminate* the hazard in the first place, for example, by designing the structural frame such that key structural columns are not located in vehicular areas, or to *reduce* the risk for any columns that remain by providing a 1 m concrete upstand encasing the foot of the column, or to *isolate* the risk of collapse by designing the structural frame to withstand the forces associated with the loss of the column.

If any given risk cannot be reduced to a low level, the designer must consider whether the proposed activity justifies the risk or whether a more fundamental design would provide a better solution that reduces the risk to a level that is demonstrably as low as reasonably practicable. Any risks that remain above the tolerable threshold must be further mitigated.

12.11 Terrorism and other malicious risks

Terrorism risks are malicious rather than accidental actions and are therefore usually considered to be beyond the scope of the Building Regulations which require the building to be constructed so that 'in the event of an accident' it will not suffer collapse to an extent disproportionate to the cause. Terrorism risks are not, however, beyond the scope of the Health and Safety at Work etc. Act 1974, which requires the designer to consider all reasonably foreseeable hazards. Clearly in some instances, for example, airports, rail stations, government buildings, designated assets of critical national infrastructure and iconic, tall or otherwise high-profile buildings, terrorism is a foreseeable risk. The same is true of other malicious risks such as safety-critical vandalism.

Terrorism, and more specifically the blast effects of a detonation due to explosive terrorist attack, is a classical low likelihood/high consequence risk, the consequences of which (should the risk occur) are often automatically disproportionate. The treatment of such risks is described in section 12.10 (Systematic risk assessment for design of class 3 buildings), and indeed a systematic risk assessment is a valid approach for considering the effects of explosive terrorist attack for all buildings where it is a reasonably foreseeable hazard, irrespective of the building risk class assigned to the building.

12.12 Achieving robustness in design

The foregoing part of this chapter has set out the principles associated with design for robustness common to all materials. The last part of this chapter considers some design aspects specific to common construction materials, namely structural steel, reinforced concrete, timber and load-bearing masonry. Inevitably it is only possible to highlight some of the key aspects associated with each material, and the structural engineer will need to consider these and other issues in far greater detail during the design process.

12.12.1 Robustness in structural steel construction

Steel-framed construction has well-developed robustness design and detailing considerations (Way, 2005) embedded into most Codes of Practice. Standard connection details are designed to sustain horizontal tie forces of at least 75 kN compatible with Class 2A and 2B design. For typical structural grids the horizontal tie forces are much larger at 300 kN or more, equating to a distributed tying force of 30–45 kN/m. One of the biggest hurdles to successful robustness design in structural steelwork is the design of sufficiently ductile connections. The slenderness ratios in steel design mean that the structure is required to undergo substantial vertical displacement before catenary action will develop. Typical simple connections can be relatively brittle with low rotational ductility that can have a capacity much less than that necessary to arrest a structural collapse following a column loss. In part this is because the standard connection details are developed considering the tie force applied in simple axial tension in the connecting beam, rather than in a combined tension and rotation associated with the catenary action mechanism shown in **Figure 12.5(a)**. The situation in the United Kingdom is further exacerbated by the fact that it is in a seismically benign region, and therefore the type of ductile detailing required in low to moderate seismic zones that increases the rotational ductility capacity of connections which assists in the arrest of collapse following column loss is not required in the UK. Consequently some connections can be particularly brittle failing at very low rotations, with fin plate connections a particularly notable example.

Successful robustness design of structural steelwork usually requires some of the other mechanisms shown in **Figure 12.5** to be developed, in particular slab membrane action (**Figure 12.5(c)**) and compressive arching (**Figure 12.5(e)**).

In Eurocode 3 (BSI, 2004), tying requirements generally follow those in BS 5950-1:2000 (BSI, 2000), but differ in two important respects. When Approved Document A extended the tying requirements to all buildings in 2004, BS 5950-1:2000 was modified and proposed gradation of tie forces for buildings of fewer than five storeys, varying linearly with the number of storeys up to the 'full' tying requirement for buildings of five or more storeys. This has been deleted from the Eurocode. Secondly, BS 5950-1 has long contained a 'deemed to satisfy' clause if the tie force was made equal to the shear force. This has also been deleted from the Eurocode.

Guidance from the US Department of Defense (2010) has emphasised the importance of ties being distributed through the slab rather than concentrated in the beams, due to the limited rotational ductility achievable under axial load in many forms of construction (particularly in steel construction). Instead, the slab is designed and detailed to provide the tying system in preference to the steel beams. BS 5950-1:2000 and Eurocode 3 are both predicated on the use of the primary and secondary beams as the main tying system, but the vulnerability of simple connections under combined axial tension and rotation led to the preclusion of the beams as the main tying system. In Class 2B buildings therefore, the primary/secondary beams should be tied as required by Eurocode 3, but it is good practice for the floor system to be designed and detailed with distributed tying as discussed with reference to reinforced concrete construction below.

Cold-formed steelwork is inherently and often markedly less robust, particularly in modularised construction. The Steel Construction Institute recommends (Grubb *et al*., 2001) that the 75 kN minimum above may be reduced to a minimum of 15 kN for discrete members or 5 kN/m where tying is distributed. Assuming that tie forces should be approximately proportional to the weight of construction and the structural grid size some reduction in tying requirements for lightweight steel construction is justified, although the theoretical derivation of the recommendation of 15 kN and 5 kN/m is unclear. Where structural spans in lightweight steelwork are substantial, the appropriate tying forces may exceed the values above and should be given careful consideration and weighed against the equivalent tie forces that would apply in conventional structural steel construction.

12.12.2 Robustness in reinforced concrete construction

The monolithic nature of *in situ* reinforced concrete construction lends itself to a robust design, although the structural engineer must still give careful consideration to the reinforcement detailing to ensure a successful the design. Both BS 8110 (BSI, 1997) and Eurocode 2 (BSI, 2004) tend towards alternative loadpath analysis as the preferred means of satisfying the requirements, although with good detailing, tie forces of 60 kN are easily achievable in RC beams and this is typical of the values specified in BS 8110 and Eurocode 2. It is worth noting that Amendment 3 of BS 8110:1997 introduced the requirement for horizontal ties to interact 'directly and robustly' with the vertical structure. This is generally achieved by ensuring that two bottom bars in each direction pass directly between the column reinforcement.

Reinforcement steel used as ties should be detailed to be continuous. Reinforcement laps should be designed for full anchorage based on the capacity of the reinforcing bar even if the tie force is lower, so that failure always occurs in the bar and not in the lap.

Reinforcement in the bottom of the RC section generally provides an enhanced tying capacity over top reinforcement, which fractures before a tension catenary can form (Merola *et al*., 2009) and can be vulnerable to being ripped out in punching shear (Mitchell *et al*., 1984). The behaviour thereafter is dependent on the bottom steel, both in terms of quantity and ductility.

A useful weapon in the structural engineer's artillery with the introduction of the Eurocodes is the ability to specify ductility grades of reinforcement. Ductility grade B (minimum 7.5% elongation) is required if more than 20% moment redistribution has been used in the design, or grade C (minimum 15% elongation) should be used for reinforcement used as ties. Minimum links are required to prevent the reinforcement being ripped out of the structure and resulting in a non-ductile failure, particularly at laps between bars used as tie reinforcement.

Load reversal acting on precast slabs may cause them to be lifted from their supports, and structural movement may cause slabs to be dislodged from their vertical support. Dislodgement of precast slabs is a particular risk when alternative loadpath analysis is used, noting the significant vertical displacements that are likely to occur. An *in situ* structural topping is required to provide structural integrity in the slab construction, with tie reinforcement provided within the topping to make the slab continuous over supports. To prevent bursting of the tie reinforcement over the support, links should be concentrated in the shear zone adjacent to each support. The robustness of precast slab construction will be optimised if the tie reinforcement is lapped with bent-up bars grouted into the hollow cores at each end of the slab units over each support.

Not all the design and detailing requirements of BS 8110-1 are incorporated in Eurocode 2; in particular, aspects of the provision of vertical ties and the anchorage of precast floor and roof units and stair members. Consequently Eurocode 2 alone is insufficient to meet the requirements of the Building Regulations and Approved Document A, and therefore those requirements of BS 8110 that are not covered by Eurocode 2 are published in PD 6687-1:2010 (BSI, 2010) as non-contradictory complementary information. Eurocode 2 requires that vertical ties are provided only in panel buildings of five storeys or more, which is at odds with Approved Document A which requires vertical ties in all Class 2B buildings (unless alternative loadpath analysis is used). Vertical ties should be continuous over the full height of the building. All precast stairs and stairs incorporated into *in situ* concrete construction should be effectively anchored to their supports whether or not they are used as ties, the anchorage being designed to be capable of carrying the weight of the unit (and imposed load corresponding to use for escape) to the tying system.

In post-tensioned slab design, the pre-stressing tendons should be fully grouted such that the risk of loss of tension in fire or explosion is eliminated. In unbonded construction, tendons should not be considered part of the tying system, and tying provided wholly with normal reinforcement. Bonded tendons provide excellent horizontal tying due to the absence of laps. Where possible, pre-stressing tendons should pass directly over the column heads between the vertical reinforcement in the columns. The tendons should be inclined such that they drop down through the slab to the bottom flange towards midspan. This gives a substantially greater resistance to punching shear and thereby greater robustness than if the tendons pass either side of the column face or outside the shear zone (Brooker, 2008; Pinho Ramos *et al.*, 2008). Where in pre-stressed construction it is not possible for the tendons to pass through the columns, the tendons should be as close as possible to the column heads and additional bottom steel should be provided to lap over the duct line. A similar arrangement should be adopted in non-pre-stressed slab construction whereby the bottom bars pass through the columns and are anchored securely into the surrounding slab. Both pre-stressing tendons and bottom reinforcement in non-pre-stressed construction should be designed for 100% of the post-failure load.

The principal remaining challenge in pre-stressed concrete is achieving sufficient interaction between horizontal and vertical ties.

12.12.3 Robustness in timber construction

In timber construction, the main design guidance is from the UK Timber Frame Association (2008). Due to the nature of timber construction, vertical ties are not usually a practical design option, and the usual approach in design is for the structure to be designed to bridge over missing elements, or to be based on design of key elements. Horizontal ties are more easily achieved: flitch plate connections in timber frame construction can accommodate the horizontal tie forces relatively easily, although splitting along the grain usually limits the capacity of the connection.

In domestic timber-framed construction, reliance is usually placed on 'effective anchorage of floors to walls' given in Approved Document A as an alternative to effective horizontal ties. Standard details are given in BS 5268-2:2002 (BSI, 2002a), although in reality these details are just a nominal connection that provides only a very modest level of robustness.

In large-panel timber panel construction robustness is typically relatively modest, while advances in timber technology have led to an acceleration in the height of large-panel timber construction (Wells, 2011) so that such buildings are now approaching a Class 3 risk classification. BS 5628-2:2002 (BSI, 2002a) recommends that internal ties should be designed for a maximum of 3.5 kN/m if distributed, an order of magnitude lower than in reinforced concrete or steel construction that, while the all-up building weight in timber construction is lower, cannot be easily justified. A continuous rim beam at each floor level capable of supporting the floor above in the event of removal of the panel below provides a continuous peripheral tie and can be an effective means of enhancing the robustness of the structure. A rim beam at eaves level in single-storey timber panel construction fulfils a similar function.

12.12.4 Robustness in load-bearing masonry construction

Historically, many load-bearing masonry buildings were exempt from robustness requirements because the height of construction was limited, but the extension of the robustness requirements in 2004 to almost all buildings changed this. Consequently, in almost all load-bearing masonry buildings except for single residential dwellings it is necessary to provide, as a minimum, effective horizontal ties or effective anchorage of slabs to walls. In Class 2B construction, the emphasis is usually on alternative loadpath analysis to demonstrate suitable bridging capacity, although in the 1970s the Brick Development Association developed design recommendations in response to the fifth amendment, showing how horizontal and vertical tying can be successfully incorporated into load-bearing masonry (Morton, 1985). More recent guidance (Brick Development Association *et al.*, 2005) gives lighter, less robust design solutions based on the standard design details in BS 5628-1:2005 described as effective anchorage rather than effective tying (BSI, 2005a), though the design solutions in Morton (1985) are preferable and closer to the intent of the robustness requirements.

It is again noted that effective horizontal anchorage achieved in Class 2A buildings using the standard design details given in BS 5628-1:2005 is very modest, these details in reality being relevant only to providing simple lateral restraint to horizontal movement of load-bearing walls, rather than to sustain the forces necessary for effective tying. The designer should implement effective horizontal tying in all Class 2A and Class 2B buildings rather than 'effective anchorage' in the former, and alternative loadpath analysis if vertical tying cannot be achieved in the latter. While this may add modest cost to the scheme, the engineer has a responsibility to demonstrate that all reasonable measures have been adopted in the design to reduce the risk of disproportionate collapse to a level which is as low as reasonably practicable.

12.12.5 Robustness during construction and demolition

Design for robustness must not overlook consideration of robustness during construction. Past events demonstrate that many failures occur during the construction phase when the structure is incomplete and demonstrably less robust, relying on temporary support for stability. In addition to the temporary support, for many hazards giving rise to a risk of collapse the risk is greater during construction than at any point during the permanent life of the building, except perhaps during demolition. The Construction (Design and Management) Regulations (2007) place duties upon the designer which are no less onerous during the temporary conditions than during the service life of the building, and the designer must consider the risk of disproportionate collapse during construction and demolition of the building as well as during its service life.

12.13 Conclusions

This chapter has intended to provide the engineer with practical guidance based on good practice in design against disproportionate collapse. The system defined in codes of practice is inevitably imperfect, seeking a balance between best-practice design for robustness, economy of design and commercial advantage between different construction materials, and the chapter has attempted to highlight some of the pitfalls faced in design and provide guidance on the design decisions the structural engineer needs to make.

The chapter is orientated towards the UK approach for structural robustness, which is now implemented in the Eurocodes and is generally recognised as being the best approach currently available and from which the approaches in other jurisdictions, where given, are generally derived. Commentary is given on the application of the different methods available for design against disproportionate collapse, from prescriptive tie-force methods suitable for low-risk structures to quantitative alternative loadpath methods for higher-risk structures. Key element design is covered as the method of last resort if the structure cannot be designed such that the loss of a given element is not disproportionate.

Guidance is given on the determination of risk with respect to structural collapse. Design for robustness is necessarily risk-based: for tall buildings and other high-risk buildings the structural engineer must therefore step beyond the limits of codes of practice and identify the actions to which the building might reasonably be subjected and their likelihood and consequences. Systematic risk assessment for the design of Class 3 buildings is described, and the application of the requirements to existing buildings is discussed. The responsible engineer must make informed decisions about levels of risk which are rigorous, consistent and not unduly influenced by other design considerations. The documents listed below expand further upon and provide additional discussion about some the issues described in this chapter.

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Chapter 13

Soil–structure interaction

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This chapter introduces the concept of soil–structure interaction and summarises the methods available for predicting the behaviour of foundations and substructural elements constructed within a soil mass. An overview of the most frequently used soil models that are incorporated into this type of calculation are discussed, with recommendations on the appropriateness of each. The development of the structural model is also introduced. Undertaking a soil–structural analysis is a multi-disciplinary exercise and therefore effective interaction, particularly between the geotechnical and structural engineers, is key. Methods to ensure this interaction is optimised are presented in this chapter. Justification of the outcome of an analysis is discussed and is extremely important. This is carried out using case-study data where possible or by means of an independent analysis. During construction, monitoring of the structural elements that have been the subject of an analysis is highly recommended. This aids in managing the risk of unanticipated large displacements on site and provides information to aid with future soil–structural analysis.

CONTENTS

13.1 Introduction to soil–structure interaction

As our towns and cities continue to grow, the number of ‘developable’ plots of land is decreasing whilst the number of constraints for those that remain is increasing. Frequently, engineers are asked to provide foundation and substructure solutions that will be constructed close to existing infrastructure, adjacent properties or subsurface obstructions such as historic foundations.

Additionally, geotechnical design is becoming increasingly complex with elements being pushed closer to their limits with respect to stress and movement tolerances. This is generally in response to achieving maximum usable space and/or reducing project costs as well as learning from our previous design works which has led to a better understanding of the performance of foundations and substructures.

Designers’ and clients’ main objectives on any project are to ensure that a cost-effective foundation solution is produced that can be constructed safely as well as ensuring that the performance of building foundations and substructures are adequate to satisfy the building’s serviceability requirements. Designers need to ensure that the foundation/substructure design and construction will not result in damage to adjacent structures and third party assets. Clients are keen to identify any potential risks early on in the design process and in particular the impact that construction activities may have on third party assets nearby.

Soil–structure interaction analyses are used as part of the design process to assist designers in identifying some of the above-mentioned issues. Soil–structure interaction is a powerful tool which can help designers gain a better understanding of the ground behaviour and hence the structure when subjected to loading and unloading. It is often used for parametric studies and to assess the sensitivity of specific assumptions such as soil parameters, construction sequencing or the overall design of a building. In the following sections, examples of projects where soil–structure interaction methods have been used to address some of the above-mentioned issues are presented and discussed.

Various relationships and methods have been developed to predict how a structural element will behave when cast against or within a soil mass. These relationships have been identified from either empirical approaches that are based on observations and case-studies or complex numerical analyses. How appropriate these techniques are to a specific problem is generally down to the judgement of the individual; however, it should be noted that the simplest design methods are often the quickest to perform and the easiest to interrogate.

For foundation and substructure solutions with an increased complexity more involved methods of calculating the forces within, and displacements of, structural elements are required. The use of the more basic approaches that rely heavily on hand calculation or ‘rules of thumb’ may not be appropriate in such situations as there may be too many degrees of indeterminacy within the problem or they may simply be too time consuming to use.

The use of finite element models allows the user to create a soil–structure model of the whole or part of a problem with relative ease (**Figure 13.1**). This model can generally then be modified to respond to changes in a design or undertake parametric studies that might otherwise take a great deal of time.

Soil–structure models can also provide us with a means of predicting the impact of one structure on another. Displacements and stress changes can be calculated for different construction phases. Such calculations are often used to justify the viability of certain developments and therefore movement is often the governing criteria when assessing the results of a soil–structure

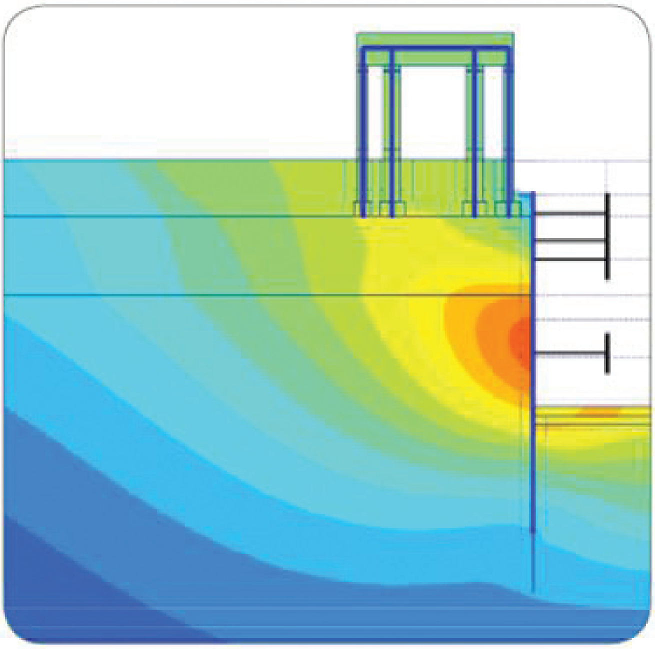

Figure 13.1 Finite element model of a sensitive structure next to a proposed excavation

analysis as opposed to the magnitude of forces within the various elements being modelled.

13.2 Methods of predicting foundation and substructure behaviour

One of the key parameters that define the performance of a structure is the magnitude of foundation and substructure movements and as a result predicting ground movement has always been the most important design aspect to satisfy. There are a number of available methods to predict how foundation and substructure elements interact with a surrounding soil mass and their complexity can vary considerably hence the appropriateness of each should be judged on:

- the quality and relevance of the information available;
- the level of detail required;
- the design stage;
- the time available to undertake the calculation/analysis;
- the sensitivity of the proposed structure and/or adjacent structures.

These various methods can be broken down into two distinct categories:

- Empirical correlations
- Numerical methods

An overview of these methods is presented in the following sections which will include suggestions of when they should be used, together with some of their advantages and disadvantages.

13.2.1 Empirical methods

An empirical method can be described as the collection of data on which to base a theory or derive a conclusion. In the field of engineering, they are generally 'rules of thumb' or simple equations that are based on a review of case-study data. It is, therefore, extremely important that the data set that has been used as a basis for a given method is applicable to a subject scenario. Examples of applications of empirical methods include correlations for the soil stiffness parameters with soil displacements of foundations and retaining walls.

13.2.1.1 Foundation settlement prediction

Accurate prediction of the settlement of a loaded foundation is one of the most challenging aspects that geotechnical engineers are asked to perform. Foundation settlement can be simplified into two distinct parts:

1. The immediate settlement resulting from application of foundation load (all soil media).
2. The consolidation and time-dependent settlement resulting from dissipation of excess water pressure in the soil (cohesive soils only).

Various empirical correlations have been developed over the years that relate the results from *in situ* testing to stiffness properties of the ground and hence foundation settlement. Some of the more commonly used are presented below.

Terzaghi and Peck (1948) produced one of the earliest empirical correlations which was based on the measured settlements of various foundations on sand. They recommended allowable bearing stress that will not result in settlement of greater than 25 mm. Burland *et al*. (1977) produced a graph which was based on observed settlement of footings on sand of various densities (**Figure 13.3**).

Later in the 1980s Burbidge gathered around 100 case records of settlement and Burland and Burbidge (1985) recommended an empirical method for estimating the settlement of foundations on granular soils.

Stroud (1989) used the Burbidge data to produce a correlation between degree of foundation loading to stiffness of granular soil and SPT value, as presented in **Figure 13.3**.

Fewer correlations have been developed for cohesive materials, as quite often the associated foundation behaviour is governed by the allowable bearing capacity of the clay rather than the settlement. However, based upon case histories of foundations Stroud (1989) provided a correlation between the drained stiffness and undrained shear strength of stiff fissured clay (e.g. London Clay). His correlation suggested that the drained stiffness is around 200 times the undrained shear strength determined from a 4 inch diameter triaxial test. Hence long-term settlement and heave of structures founded on fissured stiff clays could be estimated from elastic equations.

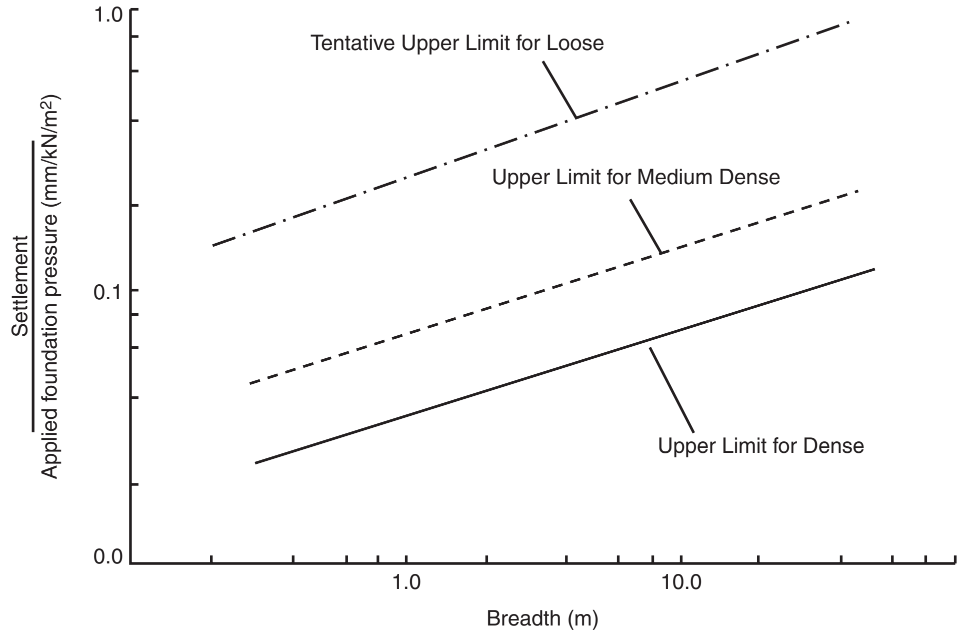

Figure 13.2 Observed settlements of footings on sand of various densities as suggested by Burland *et al.* (1977). Courtesy of J. B. Burland

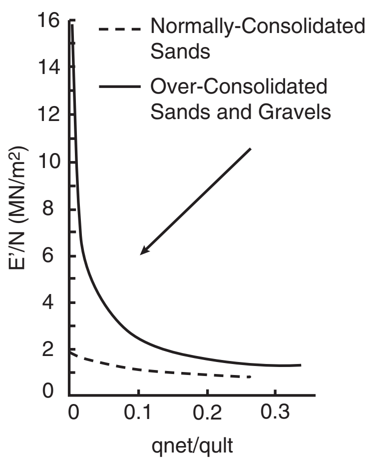

Figure 13.3 Adapted from Stroud's correlation of soil stiffness and SPT to magnitude of foundation load (Stroud, 1989)

The short-term settlement can also be estimated by relating the drained stiffness of the clay to its undrained value for an isotropic material using appropriate Poisson's ratio:

$$E_u/E' = (1+v_u)/(1+v')$$

13.2.1.2 Substructure and retaining wall movement prediction

During the initial stages of basement excavation it is common to allow the retaining walls to cantilever either the full depth of excavation or part of this depth prior to props being introduced. During this stage the soil movements behind the wall can be assumed to be at a maximum directly behind the wall with magnitudes reducing linearly with distance away from the wall (**Figure 13.4(a)**).

In the case of a propped wall, as the excavation depth increases below the prop level, deeper wall movements into the site occur. The maximum displacement within the profile of the deflected wall is sometimes described as the 'belly' and can influence deeper soil movements that may be observed at surface level at a distance approximating this movement below ground level. A typical wall profile is presented in **Figure 13.4(b)**.

There have been many published articles discussing measurements and patterns of retaining wall deflections together with the associated ground displacements behind them. These include: Peck (1969); Clough and O'Rourke (1990); St John *et al.* (1992); Fernie and Suckling (1996); Long (2001). Of these, the proposals by Clough and O'Rourke (1990) are generally used most often. The data set used to form this approach is taken from retaining walls founded within stiff clays. This method defines an envelope of vertical and horizontal ground deformations behind a retaining wall, related to the excavation depth. This envelope is presented in **Figure 13.5**.

Burland *et al.* (2001) suggest that in most cases there is a negligible risk of damage to a building suffering less than 10 mm settlement and a deformed slope (i.e. rotation) of less than 1:500. The limiting horizontal strains of a building should be reviewed separately to settlements with strains taken as the building extension over a given length, generally defined by column spacings. Burland suggests that negligible damage will occur at strain levels below 0.05%. By using the ground displacement envelopes in **Figure 13.5** it is therefore possible to predict whether a building will fall outside these limits. Should this be the case further analysis will normally be required which may include some form of numerical modelling.

13.2.2 Numerical methods

With the ever increasing ability of computers to perform complex calculations at higher speeds, predicting soil–structure displacements, soil stresses or forces within structural elements using software packages is considered a standard tool that has become increasingly the norm in engineering design.

The range of available software packages is steadily growing and varies from relatively simple programmes that integrate the sum of the values obtained from a simple formula to three-dimensional finite element packages that can simulate the interaction of complex foundation and substructure proposals. It is important that these packages are used to complement the overall design process rather than in isolation to ensure that a coherent solution is provided.

The three most common types of numerical methods adopted in solving soil–structure interaction problems are:

- elastic methods
- spring models
- finite element methods

These are discussed further below.

13.2.2.1 Elastic methods

This approach assumes a methodology whereby a soil mass is simulated using elastic theory and assigned a stiffness, generally taken as the Young's modulus. Pressures can then be applied to the soil mass which then deforms by a magnitude governed by the value of its assigned stiffness. Elastic soil models are further discussed in Sections 13.4.1 and 13.4.2.

It should be noted that there are limitations to such methods as they will not take into account issues such as the potential stiffening effects resulting from soil surcharging, changes in stiffness with time or the variation in soil stiffness with magnitude of strain.

13.2.2.2 Spring models

This relatively simple form of analysis assumes that a structural element is supported on or supporting (in the case of retaining walls) a soil mass that is simulated as a series of springs. This is often described as the subgrade reaction method. The soil stiffness characteristics are recreated by assigning each a spring stiffness. The stiffnesses may be varied in plan below a footing or raft if a 'mat' of springs is used or vertically if a retaining wall is being modelled.

It should be noted that although this method can be used to calculate forces within a structural element together with associated displacements, it will not provide any predictions of ground movements adjacent to a simulated foundation or at ground level behind a retaining wall. For embedded retaining walls there are established relationships that correlate the deflected shape of the wall to the ground settlement profile behind it as shown in **Figure 13.4**.

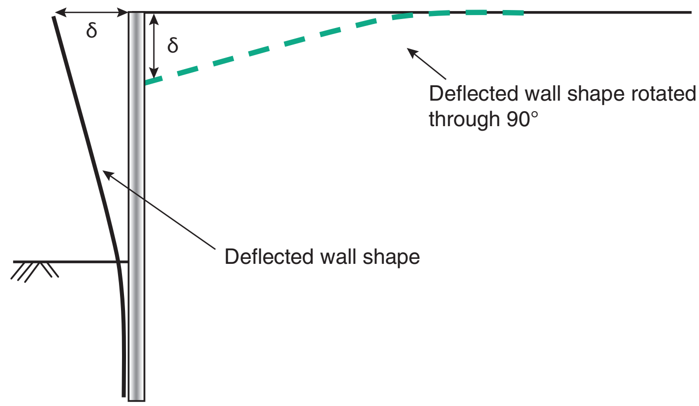

Figure 13.4(a) Theoretical ground settlement behind a cantilever wall

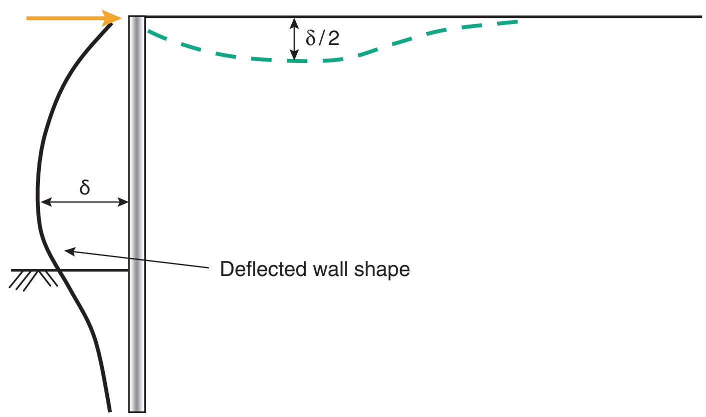

Figure 13.4(b) Theoretical ground settlement behind a propped retaining wall

13.2.2.3 Finite element method

For more complex soil–structure interaction problems where the soil mass is modelled in addition to the structural components, finite element or finite difference techniques are used. Such software packages allow construction sequences to be simulated, predict adjacent soil and building displacements and can account for consolidation and groundwater effects.

Two- and three-dimensional finite element software packages are becoming increasingly popular due to the development of their user interfaces, making the overall modelling and analysis process simpler. In theory, they have the potential to provide the 'entire solution' to a given problem. However, these tools should only be used by a suitably experienced modeller to control the quality of the results and the interpretation of the analysis. It should also be noted that there is an element of 'art' to this type of modelling; hence it is unlikely that two different engineers working on the same problem would get exactly the same results. A 'suitable' modeller is an engineer who has sufficient knowledge of the software, has sound experience or has the support of experienced engineers who can review the results both from ground behaviour and the structural response.

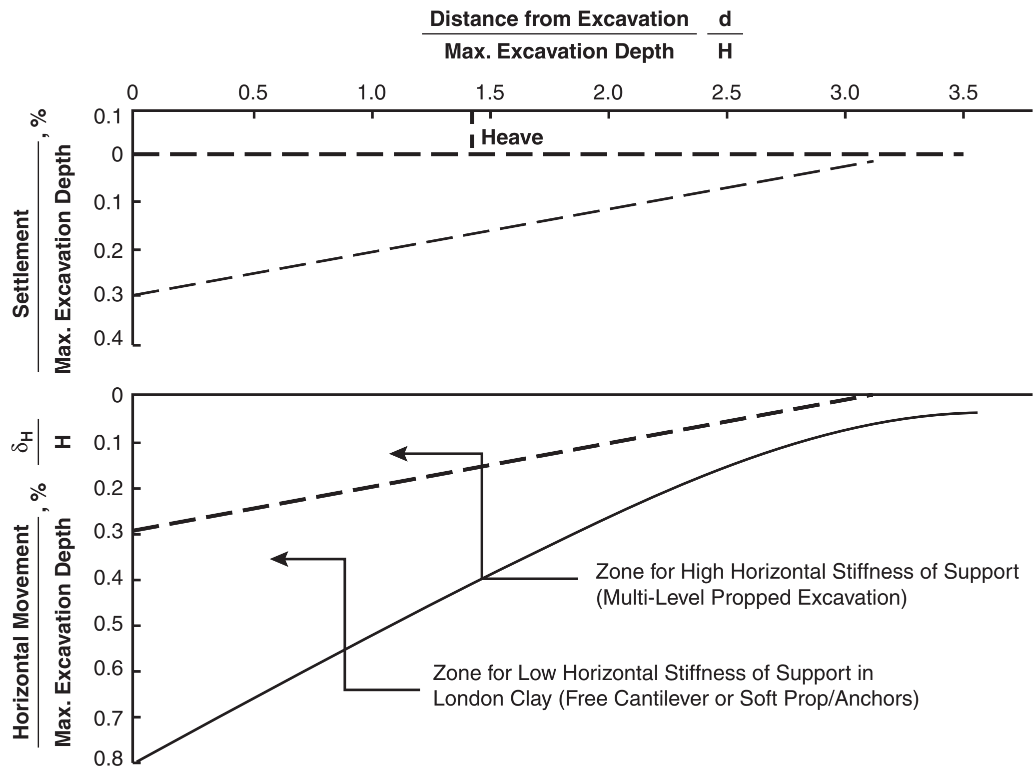

Figure 13.5 Vertical and horizontal displacement envelopes based on research undertaken by Clough and O'Rourke (1990)

Finally, it is recommended that the relevant results obtained from numerical software packages are compared with those obtained from empirical correlations.

13.3 Applications and limitations of soil–structure models

Soil–structure models have commonly been used with a view to get a better understanding of the interaction between structural elements and a founding material (i.e. soil and groundwater) and in particular predicting the resulting structural and foundation movements. These models are commonly used to assist with the design of foundations (raft, pile groups, footings, etc.) and subsurface structures (retaining walls, tunnels, anchors, etc.) as well as to quantify the level of risk associated with the impact of certain construction sequences and methods on neighbouring structures and foundations.

Appreciation of the limitations of what such models are capable of is as important as understanding their application. The following sections highlight both the instances where the use of these models can benefit a project and where their limitations preclude any worthwhile application.

13.3.1 Applications

Historically, foundation designs were progressed using established design methods that were based on case-study data, proven design formulae within standards, codes and guidance and even trial and error. As technology has improved, the use of computer software packages has increased exponentially in popularity with most graduate engineers able to use a number of products competently.

The applications of this type of software are many, with packages available that do everything from allowing the user to model the entire structural frame of a building to simulating the interaction of piles located within close proximity to each other, with the latter using some form of soil–structure interaction calculation.

When progressing the design of foundation or substructure elements the initial part of this process often involves reviewing the client's and design team's aspirations and developing a solution that responds to their scope which is both economical and can be safely constructed within a given time period.

In the early design stages there is generally insufficient information available to justify any amount of detailed analysis and therefore the various elements are assigned approximate sizes often based on experience or simple formulae. A varying level of conservatism is also exercised at this stage which relates to the level of information available at that point in time. It should also be noted that due to the fluid nature of most projects in their early stages of design (e.g. RIBA Stages B/C and possibly D) the client's brief is susceptible to potentially significant changes and therefore undertaking time consuming numerical analysis to size foundation and substructure elements and quantify likely ground movements around them may well end up being superseded.

As the project progresses, however, the design becomes much less susceptible to change and generally by the completion of RIBA Stage D the foundation and substructural options will be sufficiently established that more detailed design techniques can be undertaken. It should also be noted that by this stage a project-specific ground investigation will generally have been undertaken providing the engineer with a suitable level of information relating to soil design parameters to carry out detailed design including any associated analyses.

Soil–structure interaction applications can be considered in two or three dimensions. For example, it is possible to simulate the movement behind a retained excavation using 2D analysis and the propping effects that the corners have on a basement box can be modelled using 3D methods (**Figure 13.6**).

The elements within the ground that most commonly require the use of soil–structural models are those that either have the potential to cause surface or near-surface ground movements (and hence nearby structures or infrastructure), those that do not correspond to 'standard' design guidance and procedures such as piles at close spacing and those structural elements where load settlement behaviour cannot be satisfactorily established using simpler means (**Figure 13.7**).

It is also possible to analyse how a number of different elements interact with each other, such as the effects of loading a raft that has been formed within a basement where the retaining walls are supported by ground anchors or floor slabs. Here the behaviour of each individual element may be predicted using traditional calculation methods; however, the effect of incorporating all within a scheme may not be accounted for when considered in isolation.

Most soil–structure interaction software packages allow the user to create and manipulate the models relatively quickly. This can result in an extremely powerful tool when designing and later optimising construction sequences as the engineer has the ability to change aspects such as structural stiffness, soil properties, excavation and construction sequence, retaining wall prop heights and embedment depths allowing numerous parametric studies to be undertaken within a short space of time.

Large and complex raft foundations are also frequently modelled using soil–structure interaction techniques to predict column displacements and differential movements between the various structural and substructure elements such as cores and retaining walls. Moment contour plots showing the predicted magnitudes of forces over the plan area of the raft can also be obtained to calculate the required reinforcement quantities.

The effects of time on foundation behaviour can also be simulated using certain pieces of software. When loading or unloading a saturated clay its reaction to this change in surcharge can be said to occur in two stages:

(i) short-term – as a result of elastic deformation of the soil mass under the foundation loading
(ii) long-term – following the dissipation of porewater pressures that occurs as a result of the change in effective vertical stresses which can take many years depending on the permeability of the clay.

While with certain formulae and geotechnical software packages the short- and long-term displacements are generally estimated by the user by adjusting the soil stiffnesses within the clay layers, others are able to simulate the long-term behaviour of the soil by referring to formulae that take into account soil properties such as permeability in addition to strength, stiffness and the groundwater profile. Such software packages are also able to predict ground heave which may impact on the viability of certain substructure solutions such as a ground-bearing slab.

The ground movement and hence the movement of foundations of existing structures adjacent to a proposed development

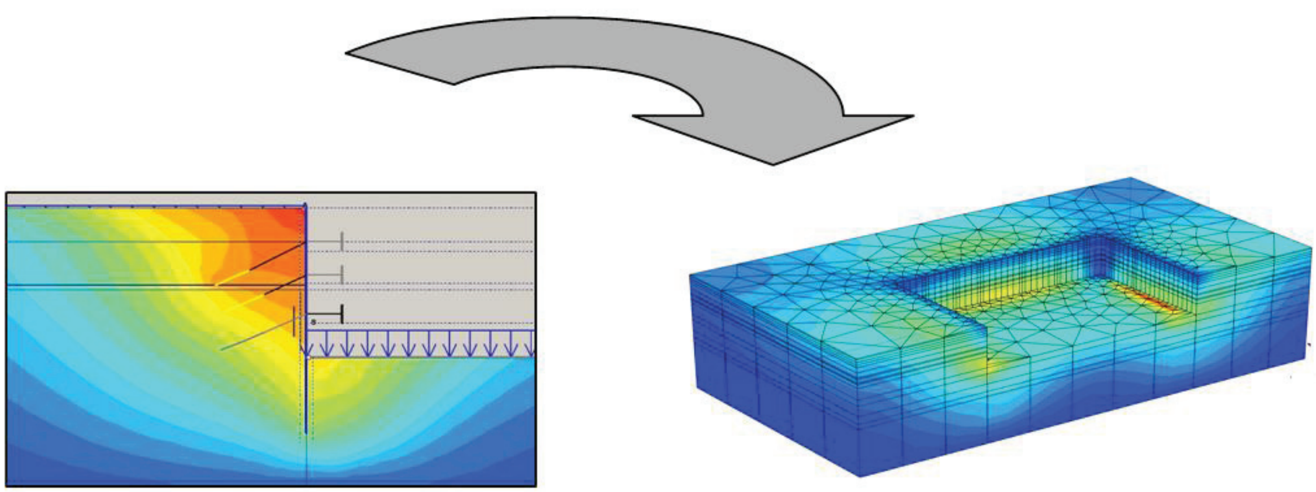

Figure 13.6 Two- and three-dimensional finite element models of a basement retaining wall

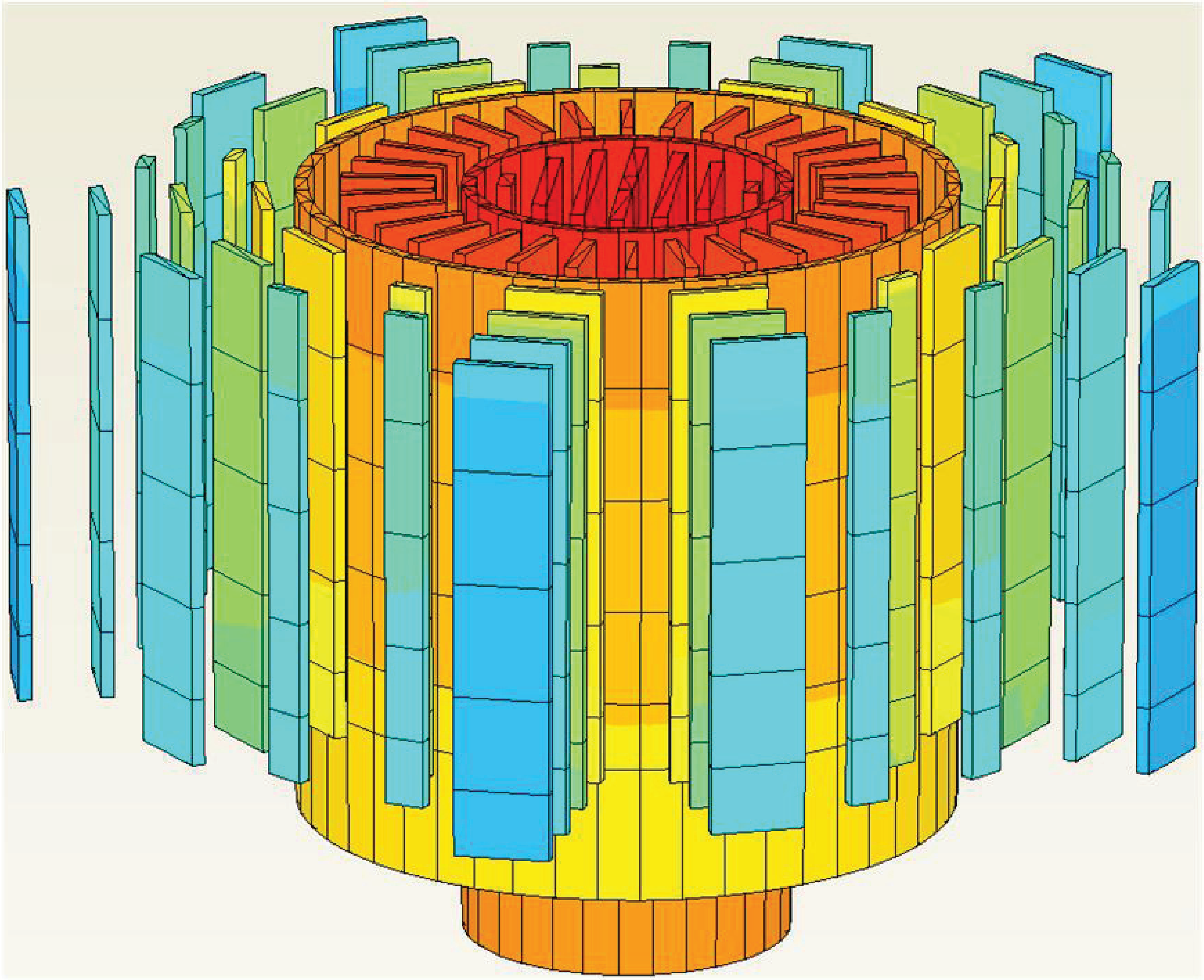

Figure 13.7 Three-dimensional settlement contours of closely spaced barrettes under a proposed tower structure

can also be predicted using soil–structure software packages. This is important when the engineer is required to quantify the risk of any likely damage to adjacent structures or infrastructure such as tunnels, roads or railways. In such studies, it is necessary to predict the movement of third party assets and hence develop a suitable construction strategy that is acceptable to all relevant stakeholders. Therefore, the competent use of these tools is a necessary requirement for most geotechnical consultants.

13.3.2 Limitations of soil–structure models

As discussed above the applications of using soil–structural models are extremely wide ranging. However, time should be spared to consider the various limitations of these models. By far, the most important limitation to note is that the model is only as good as the data that are fed into it. Even the more sophisticated models still rely on a number of assumptions based on a finite quantity of research and often the soil design parameters that are used have been obtained via empirical relationships that convert the raw data from a site investigation into the parameters required by the soil model adopted by the user.

These approximations, although based on established guidance, can lead to the user obtaining results that do not necessarily reflect the site-specific conditions. Verification of the results should therefore always be obtained prior to their issue.

It is also often the case that a software package may not be able to model either the entire design solution (possibly due to its size) or it may not be able to accurately model certain elements such as connection details leading to approximations in the geometry. These problems are widely accepted and hence models that only consider part of or a simplified version of a structure are often used and accepted during design verification although the logic behind these adjustments must be clearly documented and justified.

13.4 Ground model

13.4.1 Soil behaviour

When carrying out soil–structure analyses, various methods are available to model the ground. The selection process largely depends on the level of detail required from the analysis and the quality of the information that is available on which to create the model. Generally speaking, the more simplistic models require less detailed input data and therefore potentially provide less accurate results (although the accuracy of an analysis is by no means solely dependent on the soil model adopted).

In the soil–structure interaction analysis, the soil can either be modelled as a continuum using elastic theory with or without a failure criterion for the soil, or more crudely the soil can be defined as a series of springs representing stiffness properties of the soil only.

Soils are complicated materials that behave nonlinearly and some have anisotropic and time-dependent characteristics. Also soils tend to behave differently under initial loading, unloading and reloading, and they tend to undergo plastic (non-recoverable) deformation. The stress–strain responses of soils are both stress and strain level dependent. Such complex behaviours cannot be modelled with simple elastic models and over the years a number of soil models have been developed to represent the load-settlement and failure behaviour of soils:

- elastic soil continuum
- Mohr–Coulomb
- Drucker–Prager
- modified cam-clay
- hardening soil–model.

The elastic soil model and the Mohr–Coulomb failure criterion are the most common models adopted in soil–structure interaction analyses and these are discussed further in the following sections.

13.4.1.1 Elastic soil continuum

These models do not define a failure criterion for soil and care should be taken when used. These models treat the soil mass only as an elastic body that is generally assigned a Young's modulus, E (or Shear modulus, G) and Poisson's ratio, υ. These essentially assume that under a given pressure, over a prescribed area, the soil will deform by a magnitude dictated by the assigned stiffness values. These models also work on the assumption that if this load is later removed the soil will then completely recover contrary to the actual soil response.

When a foundation load is applied, the model calculates the settlements and stresses with a linear or nonlinear elastic soil mass (see **Figure 13.8**). Such models use established method of calculations such as the well known equations derived by Boussinesq (1885).

As mentioned above, the soil does not behave as a true elastic model and once a soil is loaded past a certain magnitude of stress its behaviour is likely to show a less stiff response (i.e. after pre-consolidation pressure has been reached). If relevant input data are available then the elastic model can be defined as a nonlinear elastic model, whereby the stiffness is defined nonlinearly and is linked to the stress level (**Figure 13.8b**).

13.4.1.2 Mohr–Coulomb failure criterion

The Mohr–Coulomb model is an elastic-perfectly plastic model which is often used to model soil behaviour. The soil behaviour is based on Hooke's law of linear elasticity for describing soil behaviour under load and Coulomb's law of perfect plasticity for describing soil behaviour at failure. The combination of Hooke's law and Coulomb's law is formulated in a plasticity framework and known as the Mohr–Coulomb model.

In the elastic range, the model's stress–strain behaviour is defined using the two Hooke's law parameters Young's modulus E and Poisson's ratio. The failure criterion is defined by the friction angle, ø′ and the cohesion, c′ as well as the dilatancy angle, Ψ′, which is used to model the irreversible change in volume due to shearing. The plastic behaviour is defined by the two strength parameters ø and c. The elastic and plastic stages are presented in **Figure 13.9**.

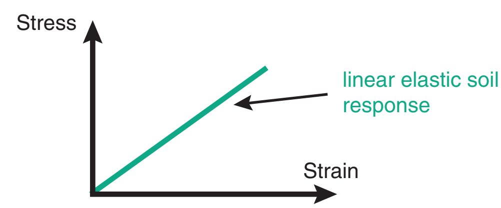

Figure 13.8(a) Linear elastic load–settlement behaviour of a soil

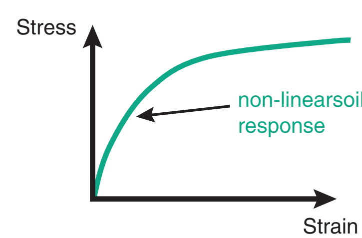

Figure 13.8(b) Nonlinear load–settlement behaviour of a soil

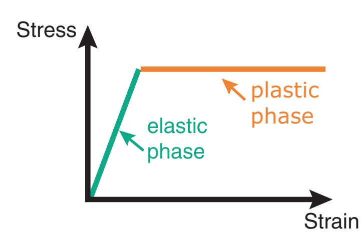

Figure 13.9 Elastic-perfectly plastic assumption of Mohr–Coulomb model

The Mohr–Coulomb soil model is relatively simple and applicable to three-dimensional stress space models and some software package models assume the soil stiffness decays nonlinearly with strain level and is generally represented by a characteristic S-shaped stiffness reduction curve as shown in **Figure 13.10**.

If the soil response to loading can be defined, these models can simulate the behaviour of a soil when subjected to structural loading very well; however, in order to achieve a good prediction accurate and sometimes quite complex soil testing is required to be carried out (e.g. stress path tests, bender element or geophysical tests; triaxial tests with small strain measurements, etc.).

13.4.1.3 The modulus of subgrade reaction model

Spring models are generally based on elastic theory and the soil mass is modelled by a number of non-interacting springs. The springs simulate the behaviour of the underlying ground at specific points rather than by a soil continuum. The benefit of this type of model is that it is generally more compatible with structural engineering software packages although no horizontal interaction between the springs is modelled.

It can be challenging to obtain and justify suitable spring stiffnesses (k) as it is defined as force over distance as opposed to pressure over area which geotechnical test results provide. Often, if springs are used to simulate the reaction of the underlying soil within structural software package, their stiffness should be manually varied by the modeller until a displacement is achieved that corresponds with geotechnical predictions.

13.4.2 Geotechnical design parameters – what is required?

Of the many aspects that one may explore when using a soil–structural model, the three most common are:

1. calculation of predicted displacements;
2. calculation of load spread from a foundation solution; and
3. ascertaining any potential modes of failure under certain loading conditions.

As a result it is very important to ensure that the soil strength and stiffness parameters adopted are as accurate as possible.

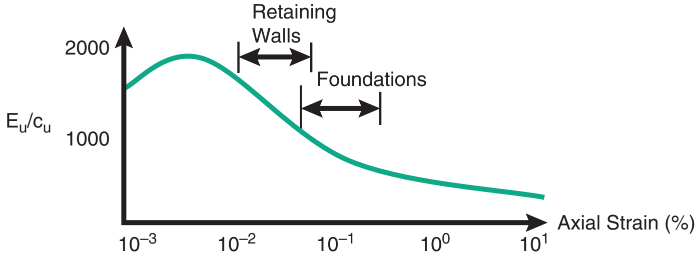

Figure 13.10 Variation in stiffness with strain as discussed by Jardine *et al.* (1985)

With respect to the latter it is often hard, if not impossible, to establish accurate soil stiffness properties for materials, as obtaining sufficient quality data from undisturbed soil samples and *in situ* testing can be problematic. It is therefore important that such parameters are derived from a variety of information that has been obtained from several different methods, including laboratory and *in situ* testing together with existing knowledge of the subject materials.

In circumstances where a high level of accuracy is required when undertaking detailed movement predictions, more complex soil models may be required. Such situations may include predicting the displacement of sensitive infrastructure or historical buildings that are located within a short distance of the subject site. These adjacent assets often have extremely stringent movement tolerances associated with them, and hence conservative movement predictions using more simplistic soil models and software are often not appropriate when looking to provide the final answer. They should not, however, be completely removed from the overall exercise as they may still provide a valuable indication of the likely outcome of future analysis during the earlier stages of design.

Research undertaken by numerous parties suggests that the stiffness of soil varies depending on the level of strain it is subjected to; the smaller the strain the stiffer the reaction of the soil. Therefore models that simulate small strain behaviour are becoming more frequently used as they are theoretically more accurate at predicting soil displacements, particularly over the initial stages of movement. Stress path tests, triaxial tests with small strain monitors as well as bender elements are the types of tests required for determining the relevant geotechnical parameters.

As previously discussed, soil models are not only used to predict ground displacements but also to demonstrate failure mechanisms and hence can be used to calculate factors of safety for elements of earthworks, retaining walls and foundations.

The results for this type of analysis depend on a series of soil strength parameters, including undrained shear strength (c_u), drained cohesion (c'), angle of friction (Φ') and angle of dilation (Ψ'). By gradually reducing any one of these parameters it is possible to discern modes of failure. Equally, this may be achieved by varying the surcharge on the underlying geology and then back-calculating a factor of safety when failure is reached.

Of the above parameters the two most often required are c_u for clays and Φ' for granular soils, which can be derived from a number of ground investigation techniques. The most common are quick undrained triaxial tests for the former and consolidated drained tests for the latter. Correlations with the standard penetration test (SPT) are also frequently used to obtain both.

The groundwater regime across a site has a large influence on the behaviour of the underlying ground and can be particularly important when modelling retaining walls and earthworks. It is important to not only define the groundwater table within an underlying aquifer but also the porewater pressure profile within cohesive material. It should be noted that this may not simply vary hydrostatically with depth, as in some urbanised areas groundwater was historically extracted for industrial purposes at depth and underdraining of the material above may have subsequently occurred.

Methods for determining groundwater levels are discussed further in the following section.

13.4.3 Site investigation design

Typically, a geotechnical site investigation is designed and undertaken around the beginning of RIBA Stage D, as by this point enough is known about the scheme and the site to allow a geotechnical engineer to predict what type and spread of testing will be required to subsequently undertake a safe and economic design and manage the predicted ground-based risks.

Where soil–structural models are likely to be used as part of the design process it is important that by the time the site investigation design is being undertaken, all the parameters that will be required for the modelling are known. Therefore, if the modelling is to be undertaken by a different engineer, it is highly important that the modeller and site investigation designer have had sufficiently detailed discussions so that the risk of data gaps within the final investigation report is minimised. If both activities are being undertaken by the same person, then that engineer should be suitably experienced in both modelling and site investigation techniques or be given suitable technical support.

When it is understood what parameters will be required the most suitable exploratory methods must be chosen. The cost of the various sampling and testing methods should be reviewed at this stage as it may be hard to justify some of the more expensive options on less complex projects where only simplistic levels of modelling are required.

When choosing the depth and spread of exploratory holes, attention should be given to the layout of the proposed building and any design constraints. A logical approach is to place the holes in the locations that will be the subject of the modelling. These may comprise areas below stability cores, areas where the site boundary borders sensitive structures or where structures or services run beneath the site. The depth of the investigation should extend an appropriate distance below the maximum anticipated founding depth. For piled foundations this may often be taken at 2 to 3 diameters beyond a pile toe. For rafts and shallow footings, depths of between 1 and 2 times the foundation width may be adopted providing the investigation extends beyond any layers that may still influence settlement predictions even at depth.

For complex analyses where high quality samples are required, the method of forming the boreholes needs to be considered. For example, triple tube core barrels may be required to produce a high standard of undisturbed sampling with appropriate laboratory testing such as small strain triaxial and consolidation tests. It is also important to complement the laboratory testing with other methods of defining the properties of a soil mass. These would likely include *in situ* methods such as pressuremeter and/or high pressure dilatometer tests, CPT tests and geophysical testing which can all be used to identify stiffness and strength parameters.

For soil–structure interaction analysis it is also extremely important to define the groundwater regime beneath a site. Standpipes and piezometers are generally used to monitor the variation of groundwater levels and porewater pressures. For sites close to tidal bodies of water understanding the tidal influence on groundwater level is also vital.

The final set of parameters adopted for numerical analysis will be based on a review of all of the test data which may potentially include some statistical analysis to justify the assumptions made.

It should be noted that the values adopted from the site investigation may be subject to change as the design and analysis process progresses. Modifications to the chosen values may occur as more information becomes available such as historical case-study data, if it contains a sufficient level of detail and is relevant to the mechanism being modelled, or field testing.

Field testing is particularly important where there is little knowledge of how a proposed foundation solution is likely to behave or interact with surrounding elements, particularly where there is a limited amount of case-study data to aid in predicting the load settlement behaviour of a foundation.

Full-scale testing may be required which may include load testing of single and multiple elements (**Figure 13.11**). Strain gauges can be included within the foundations to ascertain how the loads are shed into the surrounding ground. The results from these tests should then be compared with soil structural models with design parameters modified where appropriate in response to this new level of information.

13.5 Structural model

13.5.1 Typical structural components

There are numerous foundation and substructural elements that can be modelled in soil–structure analyses with the most common being:

- raft foundations;
- pads and strip footings;
- embedded and conventional retaining walls;
- pile foundations.

This section will discuss how different foundation types are generally modelled when carrying out soil–structure interaction analysis. It should be noted that the above list is not exhaustive and that further reference to simulating other structural components beyond those discussed below may be required.

13.5.1.1 Raft foundations

Rafts are generally modelled using finite element analysis with the raft structure represented using two-dimensional plate elements. Depending on the method of analysis the raft will

Figure 13.11 Full-scale load testing of barrette foundations

either be modelled overlying a series of springs that simulate the underlying soil mass as discussed in Section 13.4 or will be continuously supported on a soil continuum represented by a soil model. The choice of which method to use generally depends on the capabilities of the available software packages to produce the type of information, whether this be moments, stresses or displacements, required by the user. In some instances multiple software packages are required – one to calculate predicted forces within the raft and a second to predict soil response and displacements.

Ideally, given sufficient time, one would always want to model a raft using the two approaches mentioned above and then compare all the results before recommending the final set of design data.

When building the raft model it is important to include as much detail as possible. For example, the extent and location of stability cores, the lift pits, the method of connection to retaining walls (pinned, hinged or fixed), changes in level and the presence of voids should all be included in a detailed analysis.

Cores can be modelled as beams or, more ideally, as plate elements (**Figure 13.12**) and are often the most heavily loaded areas of a raft. Therefore, the highest magnitudes of displacements, stresses and moments are generally concentrated in these areas. If the core loads are applied directly to the raft without the benefit of the stiffening effects of the core walls it is highly likely that the values calculated for moments and displacements will be significantly over-predicted. This can lead to unnecessary thickenings of the raft or potentially the solution becoming unviable.

When only concerned with the design of the raft (as opposed to the structure above) it is unnecessary to model the full height of the core walls, as any increase above that of around two storeys has a negligible effect on improving the rigidity of the raft foundation below. This principle of increasing the rigidity can also extend to the surrounding retaining walls and it is vital to ensure that the retaining wall stiffness is modelled in order to obtain the correct displacement and moment profiles for the raft foundation.

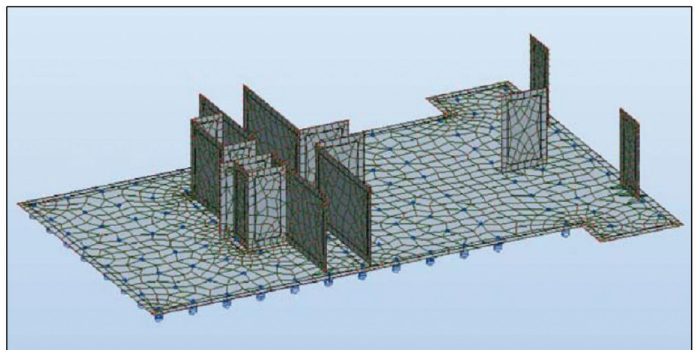

Figure 13.12 Three-dimensional image of a raft foundation model

13.5.1.2 Pads and strip foundations

Pads and strip footings are generally modelled in the same manner as a raft foundation but to a smaller scale. However, unlike rafts, there is no significant structural connection at foundation level between column locations – hence providing a much less rigid solution in the horizontal plane. When it is critical to predict differential settlements between pad locations, it will be beneficial to model each individual foundation in one analysis and, if possible, the structural frame for the two or more floors above to account for the stiffness of the overall building.

13.5.1.3 Retaining walls

Embedded and conventional (L-shape, gravity) retaining walls can be modelled in a number of different ways. Embedded walls are generally represented by beam elements, whilst conventional walls can be modelled using linear elastic soil elements with assigned properties equating to the relevant construction materials, i.e. concrete or masonry.

When modelling props, anchors or floor slabs that support a retaining wall two options are generally used – either springs or beam elements that may be horizontal or inclined (e.g. anchors or raking props) (see **Figure 13.13**).

The advantage of using springs is that it is relatively simple to calculate suitable properties. An axial stiffness is all that is generally required that accounts for the unsupported length, cross-section and spacing of the props (a slab is simply modelled as a 1 m wide concrete prop at 1 m spacings). The disadvantage of this method is that the springs react against imaginary points in space as opposed to points elsewhere within the proposed structure. Therefore, if a small basement box was to be modelled with a horizontal force applied to one of its sides only, the entire box would be held in place by the props and no estimation of lateral movement due to sliding of the box would be possible. Where such predictions are needed the props should be modelled as beam elements spanning the basement box (**Figure 13.13**).

13.5.1.4 Piles

The simplest method of modelling a pile's behaviour as a vertical support within a structural model is to assume it is a spring with an assigned stiffness, k where:

k = anticipated pile load/anticipated settlement

Settlement may be estimated using established theory such as that proposed by Fleming (1992) or Burland and Cooke (1974). Care should be taken to ensure that when estimating pile settlements, any potential group effects are accounted for. There are several commonly used pile group interaction formulae such as that proposed for pile groups in sand by Skempton (1953). Alternative methods such as modelling the pile group as an equivalent raft between two-thirds and the full length of the piles are often adopted with the depth of the assumed raft dependent on the founding conditions. The former of the two depths applies more to piles transferring the majority of their load via skin friction, with the latter associated with end-bearing groups. **Figure 13.14** illustrates the first of these two assumptions.

Software packages that are based around these methodologies generally tend to provide conservative estimates of pile group settlement.

The load settlement response of a pile or group of piles can also be simulated using finite element packages. The application of two-dimensional finite element packages in this respect is limited, hence three-dimensional finite element models are generally more appropriate. Care must be taken, however, to ensure the pile/soil interface is being modelled correctly, which requires the engineer to have a detailed understanding of the software package being used. In such situations it is highly recommended to correlate the analysis results with case-study or load test data, ensuring the soil stresses around the pile shaft and base correspond to those suggested in these tests.

With regard to laterally loaded piles, software packages are available that have adapted the theory behind embedded retaining wall analysis to allow the lateral capacity of single piles to be calculated. These are extremely useful when the lateral capacity of a pile for calculating reinforcement quantities or a

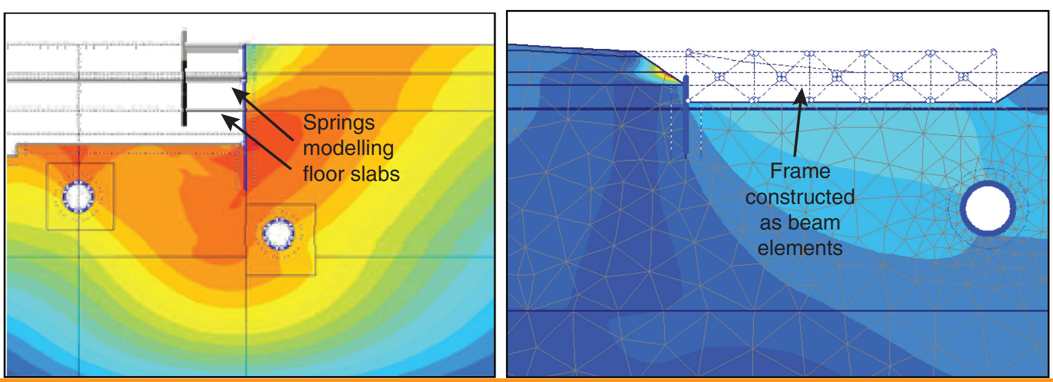

Figure 13.13 Springs and frames supporting retaining walls in finite element modelling

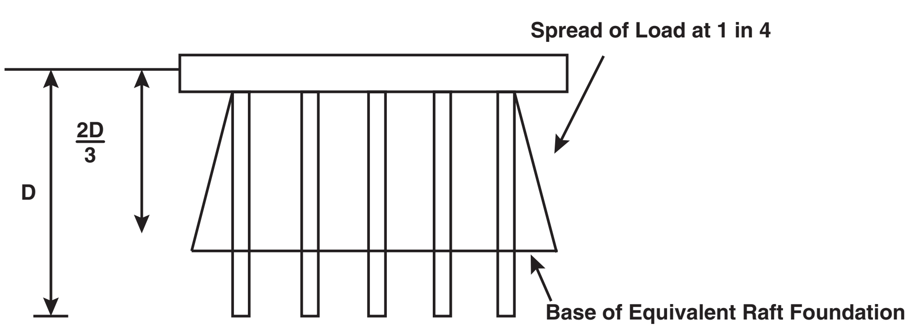

Figure 13.14 Sketch showing the 'equivalent raft' principle

predicted deflection under a given load is required. Mono-piles (subjected to horizontal loading which are not restrained via ground beams) and pile caps generally carry the highest risk in terms of exceeding their lateral capacities. Therefore, extra care should be taken to ensure that isolated piles which are not laterally restrained are not under-designed.

13.5.2 Structural properties

One of the most important properties that is required when forming a structural model is its stiffness. For most structural elements modelled, the axial stiffness (EA) and bending stiffness (EI) are generally required, with 'A' and 'I' equating to the cross-sectional area and second moment of area/m run.

Steel and concrete are possibly the most common materials to be modelled with typical values for the Young's modulus of steel generally taken between 205 GN/m^2 and 210 GN/m^2. However, choosing a representative value for concrete can be more challenging as the Young's modulus is often reduced in the long-term case to account for concrete cracking and creep. The sectional thickness, predicted deflection and age of concrete therefore need to be reviewed before a value can be adopted. Magnitudes generally range between 8 GN/m^2 and 30 GN/m^2. Thicker concrete elements that do not have long unsupported spans such as a deep pile cap spanning piles at relatively close spacings are likely to crack less and therefore would have a higher assigned stiffness than a flat slab spanning between beams and columns. A pile which is only subject to compression loads would have a higher assumed stiffness still as theoretically no cracking should occur.

Approximate guidance values for the Young's modulus, E for various reinforced concrete elements are presented in **Table 13.1**.

When defining the weight of a plate element within certain analysis packages care should be taken to ensure that no 'double-counting' occurs. In a finite element model, plates are superimposed on to the soil elements and they therefore 'overlap' the soil. To calculate the total weight of the soil and structures within the model, the unit weight of the soil should be subtracted from the unit weight of the plate. It should be noted that for plates placed along the ground surface within a model this overlap only occurs for the bottom half of the element (see **Figure 13.15**).

13.6 Developing the model with the design team

The key to developing a robust soil–structural model that accurately reflects the proposed design is to gain a good understanding of the project on a global basis. As a result, even when considering a relatively small element of the overall structure, time should be taken to understand how that element interacts with the rest of the structure and its surroundings and what constraints they will ultimately place on its design.

This will generally only be achieved via continued liaison with the project design team, with the most important parties within this group being the structural engineer and the architect. It may also be necessary to liaise with a number of external parties such as party wall surveyors and asset protection engineers from utilities or infrastructure companies. The following paragraphs summarise the key sets of information that are generally required to construct an accurate soil–structural model. Should these items not be considered there is a risk that the model will provide inappropriate or incorrect results and will not stand up to internal or external scrutiny (the latter generally being linked to external parties reviewing analyses as part of an asset protection process).

Structural element	E (GN/m^2) During construction	E (GN/m^2) Long term
Raft (>700 mm thick)	20–25	10–15
Piled wall	20–25	10–15
Suspended slab	15–20	8–12
Pile cap/piled slab	20–28	15–20
Pile in compression	25–30	20–30

Table 13.1 Guidance values for E of reinforced concrete elements

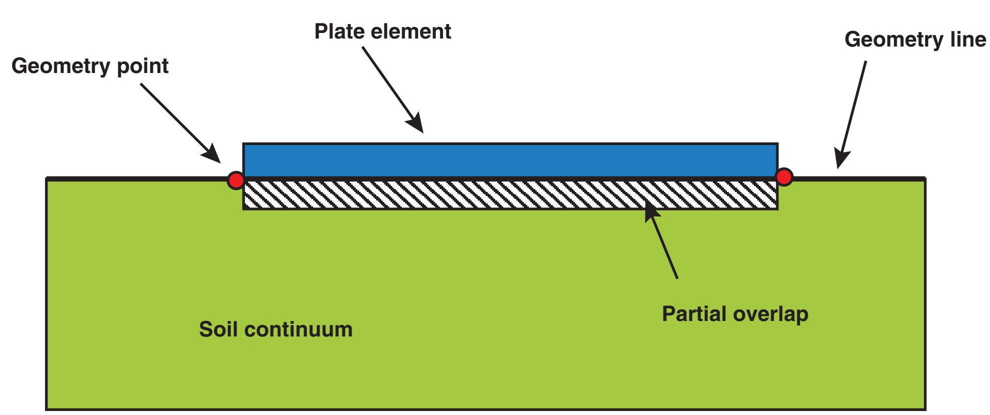

Figure 13.15 Sketch showing the overlap of a plate element placed at ground surface level

Building projects with complex ground issues require a good interface between the project's geotechnical and structural engineers. Although the geotechnical engineer will almost certainly develop and undertake any soil–structure analysis, the structural engineer will provide much of the information required.

One of the first tasks when creating a soil structure model is to review the proposed geometry of a building or structure with the best sources of information available such as the structural plans and sections including the following as a minimum:

- column locations;
- slab levels and thicknesses;
- location of stability elements;
- location of any existing structures, such as retained facades;
- retaining wall locations; and
- the extent of any voids within slabs or walls.

An annotated plan showing the various structural loads is another vital piece of information required for forming the appropriate soil–structural model. Ideally, the loads should be split into dead, live, tension, wind and any other temporary load cases, under both serviceability limit state (SLS) and ultimate limit state (ULS) conditions. Core line loads are often of particular importance as cores are generally the most heavily loaded elements.

If elements of an existing structure are to be retained it is likely that a differential movement assessment will be required between the retained portions of the building, most often elements of a facade, and any adjacent proposed structures. Items required from the structural engineer include the initial loads on the retained elements, foundation details (which can be confirmed via intrusive work if required), loading during and after construction and the maximum tolerable differential movement between the new and existing elements. Two-dimensional finite element sections taken through the retained structures are often used to aid in these assessments.

The presence of sensitive structures adjacent to or within the near vicinity of a proposed building often have a significant influence on its design, particularly when considering the new foundations and substructure. For example, the design of a retaining wall adjacent to a railway embankment may have to ensure track deflections are limited to less than 5 mm. On the other hand if the adjacent land was a 'greenfield' site the wall design would almost certainly need to only satisfy the construction tolerances. Alternatively, sufficient reinforcement can be included to withstand the moments generated within it and hence a greater magnitude of wall movement would be acceptable.

Consequently, it is extremely important to understand any external constraints imposed on the design of a building. These need to be defined early on in the design programme and in conjunction with the external asset protection engineers. Generally the maximum allowable movements need to be identified and an impact assessment undertaken to determine the risk of damage to adjacent structures. The results of the damage assessment should then be agreed as being acceptable with the relevant external party/authority.

When creating the soil–structural model the engineer needs to obtain the geometry and location of any adjacent assets in relation to the proposed foundation and substructure, an example of which is presented in **Figure 13.16**. This is generally achieved by using surveys (both intrusive and non-intrusive) and reviewing historic drawings where available. For adjacent buildings, establishing their height, foundation details, construction materials and column grid are all important elements that need to be identified. With adjacent infrastructure such as roads and railways, defining the properties of any build-up materials or embankments is as important as defining the details of the lining of an adjacent tunnel.

Establishing the magnitudes of any historic movements of adjacent assets is also important as they can be used to justify the likely impact of future movement predictions. As an example, if the displacement of a tunnel underlying a new basement is predicted to be in the region of 50 mm it may initially be regarded as being above allowable tolerances. Simulating historical loading and unloading of the tunnel associated with previous developments may highlight that the tunnel had previously been exposed to similar displacements to those being predicted. This type of information, together with justification as to the predicted integrity and performance of the tunnel in the future, will be extremely valuable when presenting design proposals to external parties (see **Figure 13.17**).

As a project progresses, the proposed programme and building geometry are likely to change. It is therefore important that any potential changes are considered when planning soil–structure analyses. Spending time at the outset of creating a model to make it as adaptable as possible often avoids the need to build new ones each time such changes occur. The use of computer-aided design (CAD) software should also be considered when looking to optimise the modelling process as most software packages allow the user to import the geometry of a structure directly into the model. This has the added benefit of reducing the risk of errors occurring if manually inputting this information.

When modelling a construction sequence one must review any available information relating to project programme, areas of phasing or any preferred construction methods that an appointed contractor may have. If the modelling exercise is required to formulate a construction sequence then it should be reviewed by someone with a suitable level of experience in construction techniques to ensure that the geometry and assumptions made are appropriate both in terms of site logistics and site safety.

13.7 Validating results

Soil–structure interaction calculations are often complex and can require a significant amount of judgement when assigning properties to structural elements or a soil mass. Consequently, there is a risk that the output may not reflect the situation being analysed. As a result, validating the output is as important as the choice of material parameters to be adopted.

The review process is key when validating a soil–structural analysis or calculation, with the first stage involving the engineer who has actually carried out the work being able to justify the methodology and the results to themselves. Subsequent reviews should be undertaken with colleagues who have an appropriate level of knowledge and experience in this type of work, ensuring that at least one person has had no previous involvement in the project. This should preclude them being influenced by any decisions or assumptions that have previously been made by the project team. External reviews should also be considered when tackling issues that are relatively new to a particular consultancy, with appropriate reviewers generally being selected based on their work on similar projects.

The validation exercise can be undertaken from a number of angles and it is recommended that several different checks are carried out with the underlying focus being on ensuring that the results make sense. One of the more effective methods of checking is to look at a particular aspect of the calculations/analysis and apply simple rules of thumb or basic engineering formulae to the same situation. If the results of the check are of the same magnitude as the analysis, then this is a positive indication that the work undertaken is producing reasonable results.

When checking the results of an analysis or calculation, and in particular the results from finite element modelling, it is recommended that simple hand calculation checks are carried out at specific locations within the soil mass/mesh; for example, checking that the vertical and horizontal stresses calculated below a raft or behind a retaining wall are of expected magnitudes. A similar exercise is also recommended associated with the groundwater pressures within the soil model. Checking that the magnitude of movements for each stage of analysis is in agreement with the trend of movements developed for

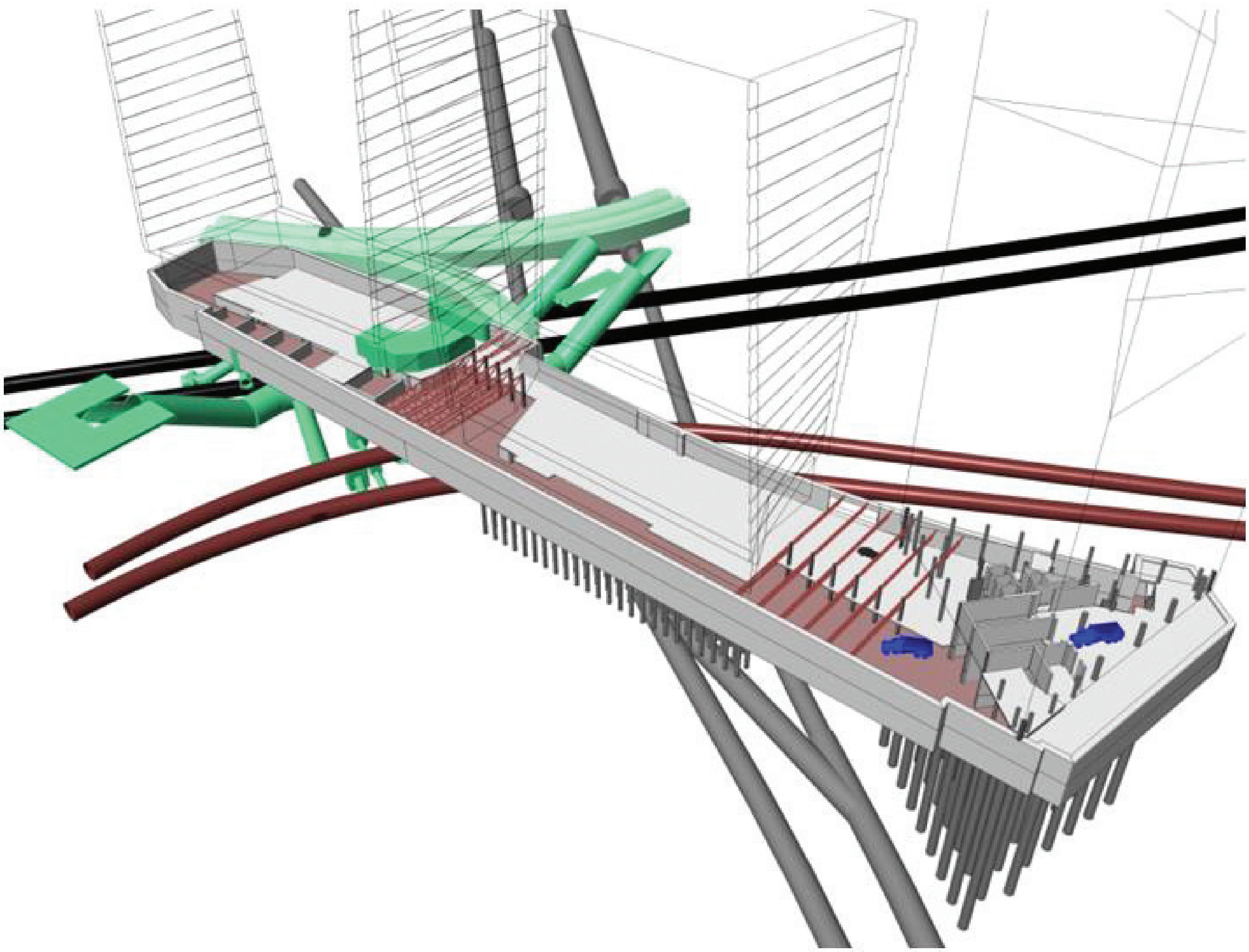

Figure 13.16 Three-dimensional model of a proposed foundation and substructure solution using information collated from site surveys, the architect, geotechnical and structural engineers

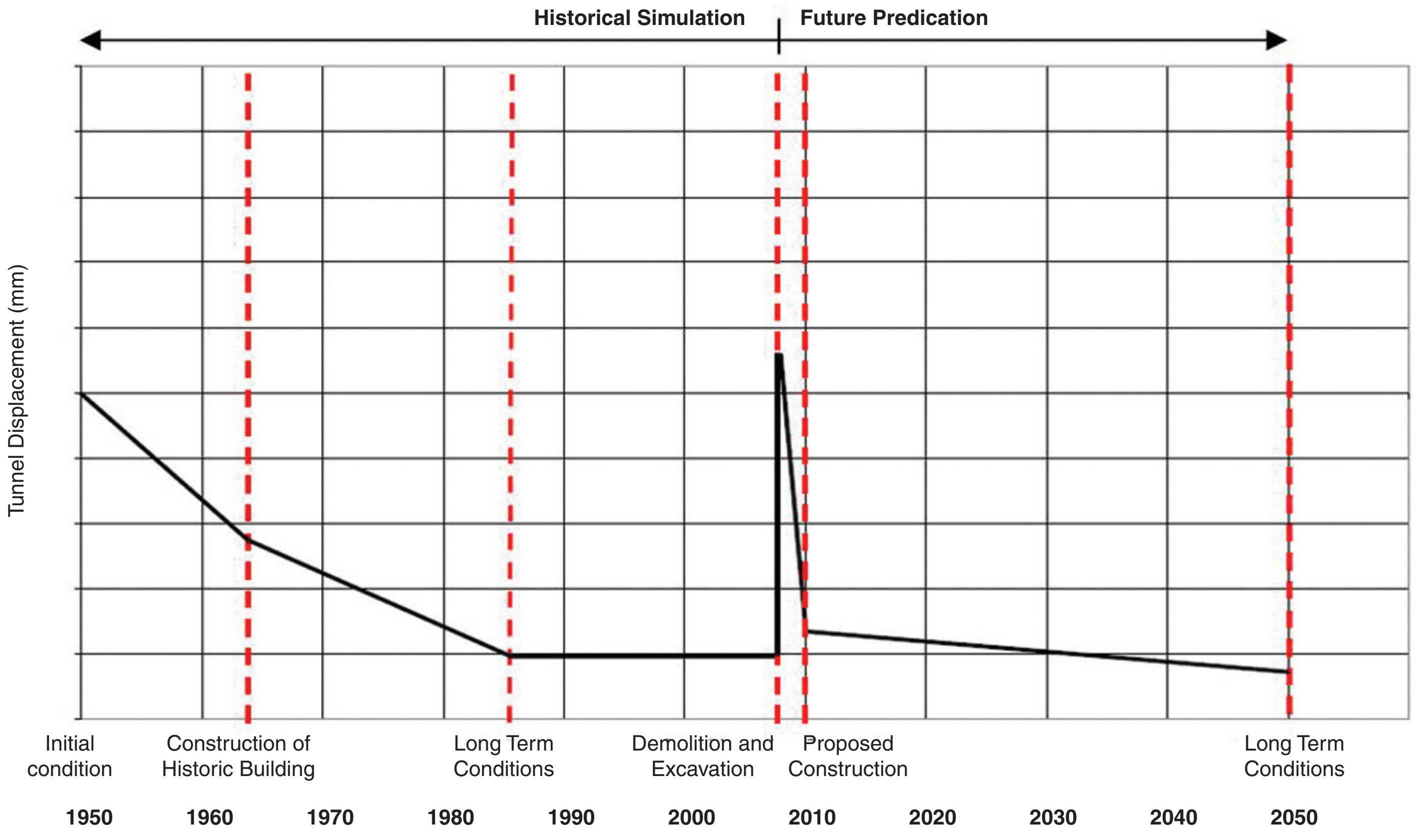

Figure 13.17 Plot of the historical and predicted displacement of a tunnel underlying a proposed development

previous construction stages is also essential, together with checking that the response is proportional to the various changes in loading.

It is also possible to replicate simple oedometer and triaxial tests using finite element software packages which, when compared with site-specific laboratory test data, can be a useful tool to check that the model is performing properly.

13.8 Calibrating the model

Being able to calibrate calculations or analytical predictions is extremely useful, especially when the engineer is required to justify their results to an external authority's representative such as the asset protection engineer or a party wall surveyor representing the interests of an adjacent property owner. Finding a relevant source to calibrate your model against could be a challenge as the industry generally does not carry out sufficient monitoring of foundations and substructures; but more importantly, we tend not to publish such results for the benefit of the engineering society. If you are fortunate enough to find a relevant case-study which relates to a project that is similar to, or that shares similar elements of, the project being worked on then your soil–structure model should be calibrated against it.

Relevant case-studies may be found in CIRIA or BRE specific publications, or technical articles in internal conference papers or technical periodical publications such as *Geotechnical Engineering* and *Géotechnique*. However, the search should always start with the incumbent designer's own database and colleagues with experience of working on similar projects should always be consulted.

An example of the effective use of case studies is the proposed development of a new commercial building that replaced an existing smaller building which overlies one of the major railway and underground stations in Central London. When predicting the impact of the developer's proposals on the underlying network of passenger and platform tunnels, it was possible to reference the historical effect that the construction of an existing building had on the displacement of the underlying underground tunnel nearby. Prior to predicting the tunnel displacements resulting from the proposed development, the construction of the existing building including the excavation of the basement was simulated using the finite element model. The soil properties were slightly adjusted until the results from the historic monitoring matched those of the analysis. These parameters were then used to predict future movements. This added a further level of credibility to the results which were subsequently used to obtain confirmation from London Underground to proceed with the design.

It may also be possible to calibrate a numerical model against factual test data obtained from a live project. A good example of such a calibration is in the case of the design of a new commercial building located within the City of London. The building is 28 storeys in height with its central core transferring very high loads to the pile cap at its base. The pile cap is founded on a number of large diameter piles that terminate within the underlying London Clay (**Figure 13.18**).

Due to the number of piles under the core and the extremely tight differential settlement tolerances that were imposed on them, a three-dimensional finite element analysis was undertaken to predict the displacement of the pile group. The load–settlement response from a number of preliminary pile load tests was also available prior to the commencement of the piling works. These tests were also modelled using the finite element software, with the parameters adjusted until reasonable agreement with the field testing was reached. These parameters were then fed into the pile group analysis which predicted settlements that compared extremely well to those observed on site (see **Figure 13.19**).

13.9 Monitoring

Monitoring the displacement of structures and infrastructure in and around a construction site is often required as part of a party wall or asset protection agreement (**Figure 13.20**). Even though it is widely recognised that the ability of soil–structure models to predict movements is improving, they are by no means infallible and hence the inclusion of a monitoring regime is a common method of managing the risk of extreme and potentially damaging displacements occurring.

The benefit of monitoring critical structures during demolition, excavation and construction, is that trends in displacement can be observed which will provide advanced warning prior to significant levels of damage occurring. The points at which displacements are expected to cause damage are ascertained during a project's design phase. Trigger levels are then calculated which act as indicators of how close a structure is to critical levels of movement. Typically 'amber' and 'red' trigger levels are

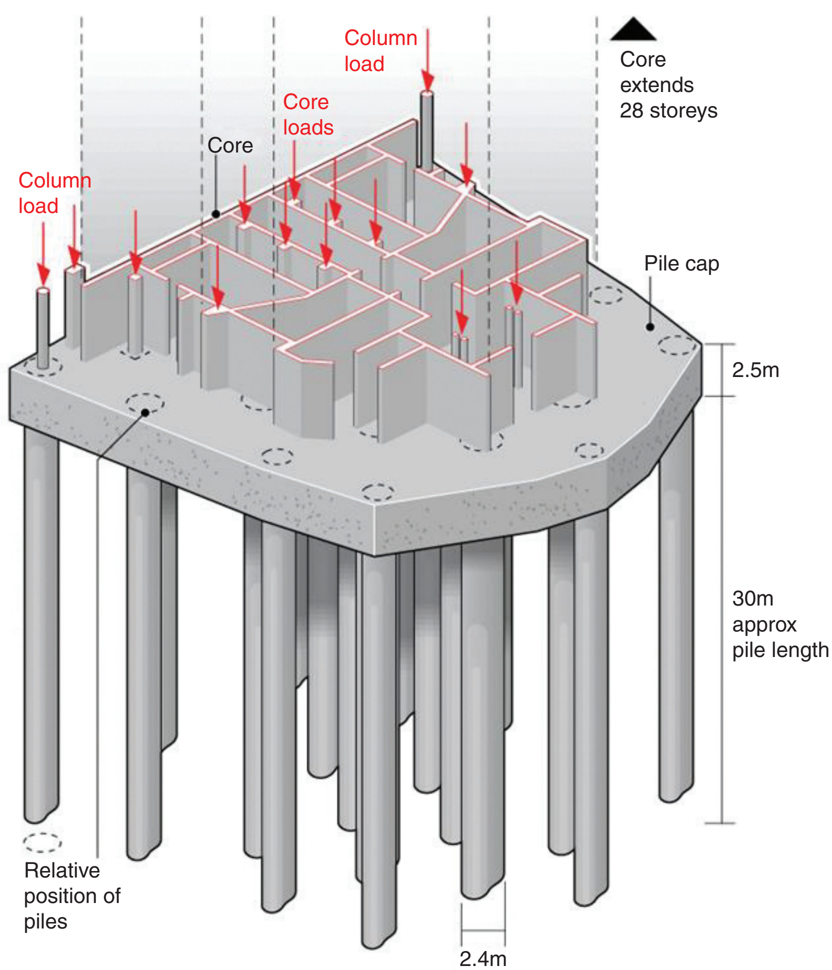

Figure 13.18 Image of the core foundations of a 28-storey tower

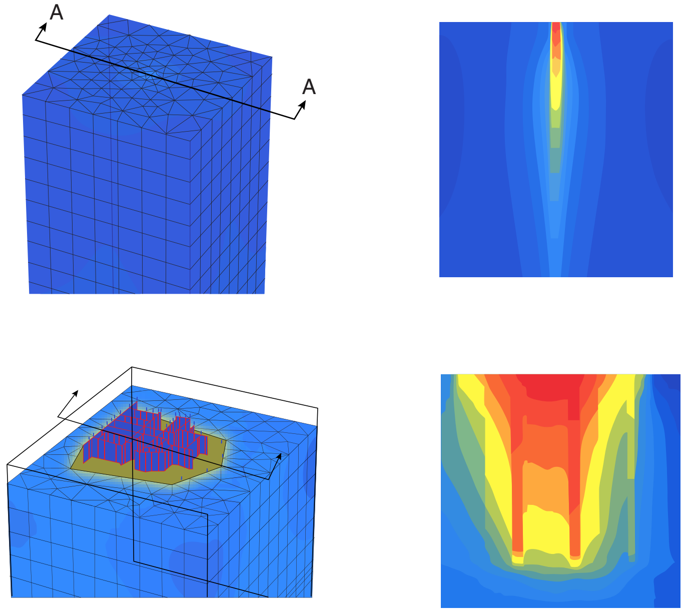

Figure 13.19 Three-dimensional finite element analysis of a pile load test and correlated results of the group settlement

Figure 13.20 Installation of automated monitoring equipment within a London Underground tunnel underlying a new commercial development

assigned to predefined magnitudes of movement that correspond to a percentage of the maximum tolerable displacements. Actions are assigned to each level which may vary from 'organising a review' to 'complete cessation of all work on site'.

The trigger level system allows the engineer to re-evaluate the movement predictions from their soil–structure interaction model as the construction process progresses. For example, if retaining wall movements are in excess of predictions for a given stage of a basement excavation the propping strategy may need to be modified. Similarly the soil–structure model could be adjusted to revise the predicted movements for the next stage of construction. This provides a sometimes critical level of control during the construction of projects within close proximity to sensitive structures. A typical definition of trigger levels for a project is presented in **Table 13.2**, together with example actions required from the monitoring specialist and contractor on site.

The review of monitoring results also brings significant benefits to the engineer and in some cases the developer, particularly on phased projects. In situations where a multi-phased development extends along the route of an underlying tunnel, next to an adjacent road, railway or series of sensitive structures, the monitoring results observed during construction of the initial phases can be used to justify the predictions of later phases. Without this opportunity, and with a lack of site-specific historical knowledge, an element of conservatism is often applied to the soil–structure modelling process to reduce the risk of under-prediction. This can lead to elevated construction costs that could otherwise have been avoided if site-specific data had been available.

Monitoring can also be used to protect a developer. For example, if a claim is made by a third part asset/adjacent property owner that construction activities resulted in the cracking of a nearby structure, the results of the monitoring can often be used to check whether the on-site activities and movements were of such magnitude that they could have resulted in the claimed damage.

Even with the advantages listed above, there are still limited amounts of movement monitoring undertaken as the short-term cost savings are often judged to outweigh any long-term benefits.

Action level	Alert status	Ground monitoring specialist action	Contractor action
Green	Minimum alert status which indicates that the instrument is functioning correctly and within all allowable levels	No action required	No action required
Blue	First alert status, no action required on site. If an instrument reaches a blue alert status then the instrument should be closely monitored to identify further trends in movement	Pay particular attention to the monitoring results of those monitoring points that exceed blue trigger level	No action required
Amber	Action required on site to avoid further deformation. This action level indicates a movement approaching the maximum levels included within the relevant specifications. Contractor to prepare emergency action plan	Manual survey of adjacent buildings infrastructure to confirm real-time results. Increased frequency of manual monitoring locations	Emergency action plan to be defined by the contractor for red trigger level to explain how any breach will be mitigated. The contractor shall implement action plan for breaching amber trigger level
Red	Action required on site to stop/minimise any further movement. This action level indicates a movement beyond the levels included within relevant specifications. The design team should review all results to assess the impact of such movements on the adjacent structure	Maintain increased monitoring frequency to ensure that mitigation measures are effective	Contractor to stop work while design team review results. If agreed by design team, emergency action plan to be implemented by the contractor

Table 13.2 Typical trigger level definitions with associated example actions

Many of the more common methods of predicting soil–structural movements are based on the back analysis of historical monitoring data. As more information becomes available across the industry our experience and confidence in making movement predictions increases, allowing less conservative and more economical and sustainable designs to be produced. Movement monitoring, particularly for projects that involve the construction of unusual structures or that are located close to sensitive buildings or infrastructure, should therefore be encouraged by engineers as it provides vast potential benefits across the entire industry.

13.10 Conclusions

Soil–structural interaction is a powerful tool and can help designers gain a better understanding of the ground behaviour and hence the performance of a structure. Judgment of the engineer in using an appropriate method of analysis at the right stage of design is key to achieving the desired accurate outcome in a timely manner. Soil–structure interaction analysis is the best method of predicting the magnitude of foundation and substructure movements as well as assisting the engineer in understanding the ground behaviour under loading and/or unloading. The quality of results obtained from such methods of analysis is highly dependent on the input data from which the ground model is created. This in turn relies on obtaining appropriate site investigation and geotechnical testing carried out to define the appropriate engineering properties of the ground.

A number of examples of numerical models presented in this chapter show the range of application of soil–structure interaction in the industry. The best results are often obtained when the model is jointly developed with the design team, and in particular the project structural engineer, so that an in-depth understanding of the structural elements is achieved and the most appropriate cross-section is modelled in the case of a 2D soil–structure interaction study. Where possible, the model needs to be validated against case-study data. Also the engineer must interrogate the output from numerical analyses and be satisfied that the results are sensible (e.g. check magnitude of vertical and lateral stresses and changes in water pressure).

Finally, results obtained from soil–structure interaction studies are generally based on a number of assumptions. The significance of any predictions dramatically increases if they are confirmed by movement monitoring results of the structures that have been modelled. Where possible, the engineer must ensure and encourage monitoring of: the structures during and after construction; below-ground infrastructures such as tunnels below foundations; buildings adjacent to deep excavations; and rafts or piled raft foundations of complex and tall buildings.

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Chapter 14
Materials

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Materials of suitable strength, stiffness, flexibility, durability and affordability are key to the realisation of good design. Furthermore, in a world greedily competing for scarce resources, it is essential that the use of materials is economic and that a high degree of recycling is achieved. This chapter gives advice on masonry (including ceramics and stone), metals (cast and wrought iron, steel and aluminium), concrete, timber, glass and polymers. It is hoped that the content will give a satisfactory grounding for designers to achieve a working knowledge of some of the most frequently used materials. The chapter gives advice on problems that can occur with the careless use or inadequate protection of materials. Different types of metal corrosion are listed and ways to minimise their effect. Concrete, and in particular reinforced concrete, has suffered from a variety of problems such as carbonation, alkali silica reaction (ASR) and misuse of high alumina cement (HAC) – these are explained and, where necessary, advice given on elimination or minimisation of problems. Where appropriate, ranges of mechanical properties are listed. Additionally advice is given on performance in fire and the protection of materials to resist fire.

CONTENTS

14.1 Introduction

The aim of this chapter is to provide engineers involved in structural design with a basic knowledge of those materials most frequently used in new construction and refurbishment. The topics covered include masonry (including ceramics and stone), metals (cast and wrought iron, steel and aluminium), concrete, timber, polymers and glass.

14.2 Masonry

14.2.1 Ceramics

14.2.1.1 Introduction

The word ceramics has its origin in *Cerami*, the potters' district of ancient Athens. In simple terms, ceramics is burnt clay. The fact that some 3500 year-old masonry structures still exist is testament to the durability of the material if manufactured and constructed to high standards. For the purpose of this section, the key ceramics are bricks, blocks, tiles, vitrified clay-ware, terracotta and faience. Although not of ceramic origin, for the sake of completeness, a note about calcium silicate bricks is also included.

The demolition material from ceramic masonry structures can be recycled by cleaning off mortar for reuse as brickwork or crushed to form aggregate for low strength cementitious materials. A lucrative business exists for the sale of London stock bricks to be used in refurbishment schemes and in the repair of existing structures.

Brick masonry walls have exhibited excellent fire resistance provided that any supporting structure maintains integrity for the duration of the fire. In the absence of information from UK Codes further guidance on performance in fire can be gained from Edgell (1982) and de Vekey (2004).

14.2.1.2 Bricks and blocks

These are probably the earliest unit of industrialised construction dating back to about 1300 BC. Examples of that construction still exist in Choga Zanbu Zigorat. In the early days, brick-making was usually a parochial affair with manufacture often taking place on the construction site. This practice has led to a wide variety of types of brick the governing factor being the suitability of the local clay. In early times clay was moulded into blocks and then allowed to dry in the sun before use. UK practice distinguishes between bricks and blocks by face size. A unit smaller than 300 mm × 100 mm is a brick, larger sizes are blocks.

Bricks for use in structural situations are provided with frogs (a shaped indent to the top of the brick). These bricks should be laid frog uppermost with the frog filled with mortar to provide adequate wall or column strength.

Architects, engineers and bricklayers should be familiar with the different systems of brick/block bonding to ensure it is appropriate to the requirements of the construction under consideration. These include *stretcher*, *header*, *English*, *Flemish*, *garden wall* and *monk* bond. These are illustrated in **Figure 14.1**. In Pakistan *Quetta* bond has been used to increase resistance to earthquakes.

Mortar (essentially a mixture of cement, sand and water) used in construction of brickwork should comply with BS 5628-3. If good quality work is required then the use of quality controlled, ready mix mortar delivered to the site is preferable to site mixed material. Colouring agents may be added to the mix to achieve architectural preferences. There is also widespread practice of adding plasticisers and/or retarders to the mix to control workability and setting times.

Stainless steel or other non-ferrous bed joint reinforcement may be added in special applications such as gable walls to houses or perhaps where openings occur in order to supplement other means of support. Several proprietary brands of reinforcement are available but are usually of welded fabric configuration available in coils. For some structural applications,

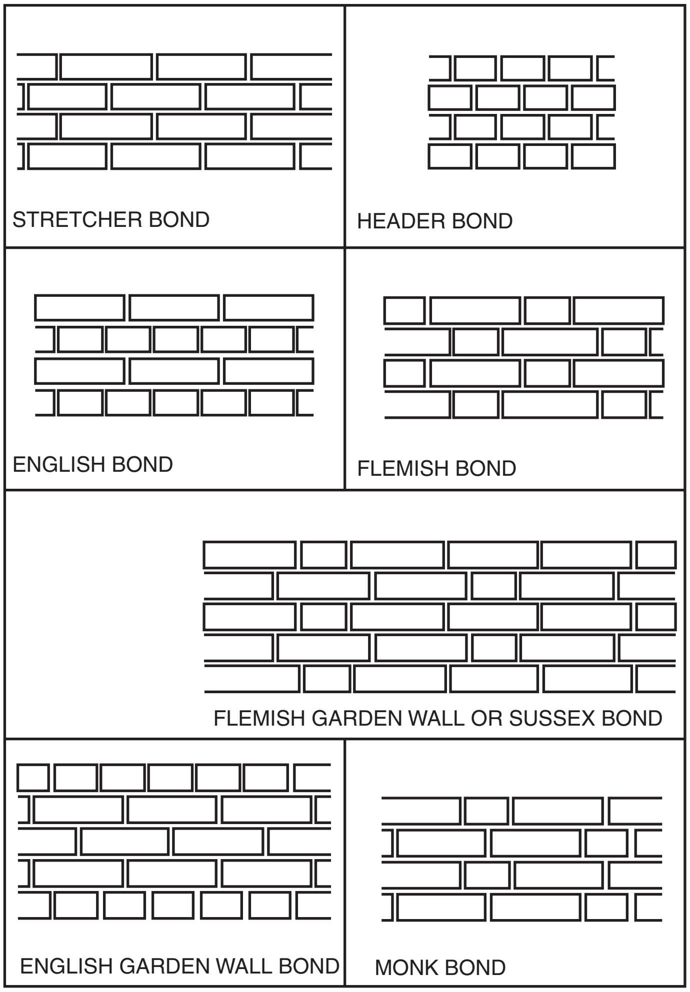

Figure 14.1 Brick bonding (Doran 2009). Reproduced courtesy of Whittles Publishing

perhaps where lateral loading is to be resisted, masonry may be pre-stressed.

Bricks and blocks may be solid or perforated (to reduce weight and/or increase thermal properties). The compressive strength of bricks varies enormously from about 7 N/mm^2 to well in excess of 100 N/mm^2 for clay bricks and 21 N/mm^2 to 60 N/mm^2 for calcium silicate and concrete bricks.

Cavity wall construction is usually used to enhance insulation. Such walling may consist of *brick and brick* or *brick and block*. The two leaves should be tied together with non-ferrous ties to increase stability and load-bearing capacity. With the need to better conserve energy a number of systems for increasing the thermal capacity of cavity walls a number of systems have arisen which include:

- The inclusion of compressed glass-fibre batts within the cavity during construction.
- A retrofitting injection of polystyrene pellets or foam into the cavity through holes drilled in the external skin.

A major concern which affects the efficiency of cavity construction is the carelessness of bricklayers who allow mortar droppings to fall to the base of the cavity during wall construction. This should not occur but if it does, unwanted material can be cleared out using the technique shown in **Figure 14.2**.

Brick suppliers such as Ibstock (www.ibstock.com) supply excellent information sheets indicating brick dimensions and quantities to cover specific areas.

Common bricks

Although the term *common brick* is frequently used in the industry, it finds no specific resonance in British Standards. They are simply bricks of sufficient strength and durability to be used in situations where they are not permanently exposed to view.

Facing bricks

These are bricks that exhibit a pleasant appearance when used in situations where they are permanently on view. Many different types are available and can be viewed on display at brickmakers' premises or in trade catalogues. Where appearance is important, practitioners should be encouraged to arrange for display panels to be constructed before making a final decision on types of facing brick.

Engineering bricks

This is a term of convenience in the UK but does not appear in British or European Standards. However the National Annexe to BS EN 771-1 does use the term. Generally speaking these bricks have lower water absorption but higher compressive strength characteristics than other bricks and are suitable for use in aggressive environments. Typical uses are for damp proof courses or at the bases of freestanding masonry retaining walls in order to eliminate the plane of weakness caused by sheet material damp proof courses. Best practice suggests the use of two courses in these circumstances. BS 5628-3:2005 gives guidance on the use of these and other bricks for use as damp proof courses.

Other bricks

The range of other available bricks is very wide. Their descriptions are often related to the process of manufacture and include:

- extruded wire-cut
- pressed
- soft-mud
- stock
- hand-made
- specials
- stiff plastic
- semi-dry
- clamp
- burnt.

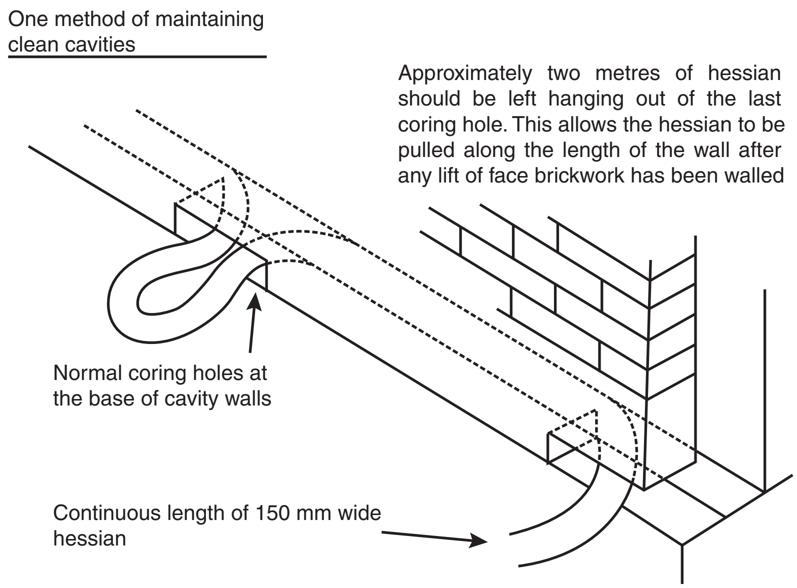

Figure 14.2 Removal of mortar from cavities (Doran 2009). Reproduced courtesy of Whittles Publishing

14.2.1.3 Calcium-silicate bricks

Although *not* ceramic in nature, but for the sake of completeness, the following information about calcium-silicate (sometimes known as sandlime or flintlime) bricks has been included. These are made from lime, silica, sand and water. As an alternative, crushed or uncrushed siliceous gravel or crushed siliceous rock is sometimes used instead of, or in combination with, sand. Colouring agents are added and then the materials are pressed into shape and subjected to high pressure steam autoclaving. Calcium silicate bricks are more susceptible to shrinkage than ceramic bricks. Arises are particularly susceptible to damage so particular care in handling is essential.

14.2.1.4 Tiles

Roof tiles

Roof tiles are usually moulded and fired clay made using Eutruria clay. Roof tiles can be conveniently categorised as plain, interlocking or classical.

Plain or interlocking tiles are also known as single or double lapped tiles. The standard size for a double lapped tile is 265 × 165 mm, usually 12–15 mm thick and provided with lugs or a continuous rib. They can be single or double cambered to suit particular architectural choice. Each tile should be supported on a timber batten and this support supplemented by nailing at the head of the tile, or by clipping, to resist wind suction. Areas of roofing are regularly stripped off in inclement weather. The great storm of 1987 exposed many roofs where original metal fixings had badly corroded, thus making the case for the use of non-ferrous metallic fixings.

It is generally accepted that roofs using these tiles are waterproof for pitches above 35° although some manufacturers claim that this can be as low as 30° or less.

The term ‘classical tiles’ is a general term used by many manufacturers to indicate a history dating back to Roman times. They are generally plain tiles of 265 × 165 mm dimension and ribbed to locate with tiling battens. Another version termed ‘peg tiles’ is un-ribbed but provided with holes to accommodate nail fixings. Classic tiles are recommended for minimum pitches of 35°. Alternatively they can be used in near-vertical tile hanging situations. The normal material is clay although modern facsimiles are now made in metal.

Wall and floor tiles

Clay-based tiles for internal or external use may be glazed or unglazed with smooth, textured or profiled surfaces. Tiles for external use must be frost resistant. Usually square or rectangular the face dimensions range from 100 mm to 300 mm with thicknesses in the range 5.5–8.5 mm. Specials, however, may be up to 300 × 600 and up to 30 mm thick and non-rectangular in shape. Tiles from earlier times often change hands for very high prices causing difficulty in repair and refurbishment work.

The effectiveness of fixing depends crucially on the preparation and quality of the substrate. Bedding material may be cement/sand mortar or modified cement/sand mortar in which synthetic resin emulsions such as styrene butadiene are used to enhance the adhesive properties of the mortar. After fixing, except where specifically designed movement joints occur, low-shrinkage grout material should be used to fill all joints.

Success in the use of mosaic tiles as a decoration for high-rise blocks is heavily dependent on good workmanship to achieve durable results.

14.2.1.5 Pipes

Although the market is perhaps dominated by plastic pipes, cylindrical clay pipes of 100–600 mm bore are used for domestic drainage and other applications. These can be supplied in 200, 300 and 1000 mm lengths with spigot and socket factory-fitted push fit flexible couplings. These units are of vitrified clay fired at temperatures of 650°C to 1100°C which gives the material a glass-like surface. In the UK, pipes are usually made from coal measure shales. A variety of bends, bell-mouths, adaptors, gullies and junction pieces are available. In the modern era the analysis of faults in underground drainage is greatly enhanced by the use of closed circuit television. In some situations, it may be possible to insert new plastic liner tubes rather than the more expensive technique of replacement.

14.2.1.6 Terracotta and faience

Terracotta (see **Figure 14.3**) is normally a red coloured, dense, hard ceramic formed from once-fired clay. The reddish colour derives from the Etruria clay from which it is made. Alternative colours such as buff and dark slate are available and can be achieved using different types of clay. A variety of shapes are produced by highly skilled hand moulders from the wet clay and cast in plaster or rubber moulds. The finished products may be used in decoration, cladding, ornate chimney pots, finials and other applications. Although the use of terracotta dates back to the mid-1800s (and perhaps as early as Roman times) architects are still using the material in modern designs. An example of this is the use of ventilated rain-screen cladding by Renzo Piano for the Potsdammer Platz in Berlin.

Faience is terracotta that has been glazed prior to a second firing. It is usually made into decorative panels which provide attractive decorations to the elevations of buildings. Some units are profiled to facilitate a good key to backing material. For tall buildings, it is customary to use copper or stainless steel securing wire cramps at intervals. Because of potential problems with drying shrinkage it is prudent to limit sizes of panels to 450 mm × 300 mm or 300 mm × 225 mm.

14.2.2 Stone

14.2.2.1 Introduction

The dominant stones for construction purposes are:

- **Igneous.** These have crystallised from molten rock or magma. In the UK the most frequently used igneous rock in construction is granite, a coarse-grained material containing at least 66% silica consisting mainly of quartz, mica and feldspar. These rocks are very resistant to weathering due to their low porosity. Granite is typically found in Cornwall, Cumbria, Ireland and Devon. As with other types of rock, a great deal of material is imported through Italy although much of it originates in Portugal, Spain, Turkey or Greece.
- **Metamorphic.** The origin of these rocks is that they have been produced from re-crystallised sedimentary rock. Examples in common use are slate and marble. Marble is typically found in Connemara and Ledmore. Material described as Purbeck marble is in fact a limestone which is capable of being polished. Slate is quarried in Wales, Cumbria and Cornwall and is also imported from Spain, China and Brazil.
- **Sedimentary.** These are the main UK building stones. They have their origin in a two-stage process. For sandstones, sediment is initially deposited; secondly, the compaction pressures from movements of the earth's crust produce a hard rock. The cementing agents may be siliceous (containing silica), calcerous (containing calcium carbonate) dolomitic (containing dolomite and/or magnesium), ferruginous (containing iron oxide) or argillaceous (containing clay). These factors combine to give these rocks their individual character and durability.

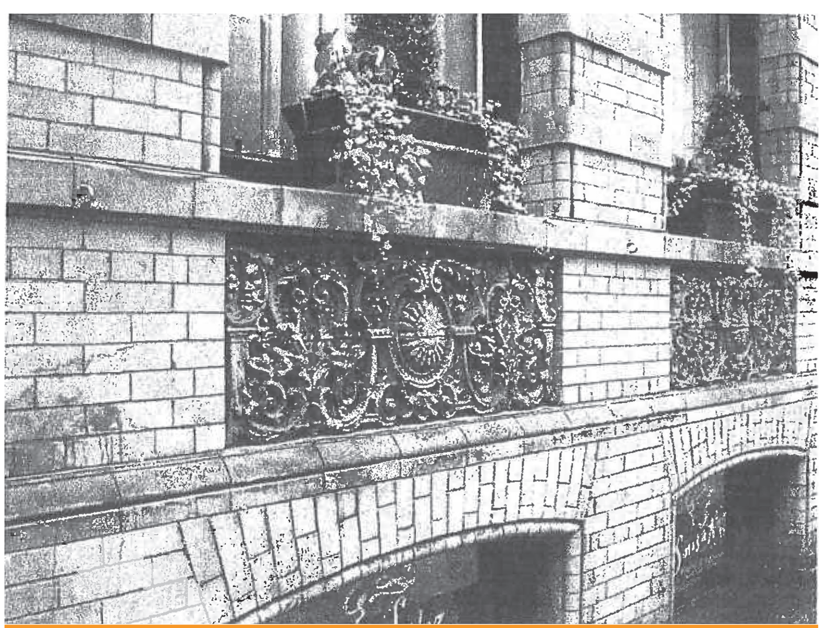

Figure 14.3 Terracotta. Courtesy of Dr Geoffrey Edgell

Limestone sediments are usually formed from the skeletons and shells of aquatic animals or from chemically formed grains such as ooliths.

14.2.2.2 Extraction

The term *plug and feathers* is used in the British Isles for the quarrying of large blocks of stone. This refers to a technique where metal plates (feathers) separated by wedges (plugs) are inserted into a series of holes drilled into the rock. All wedges are driven in simultaneously to split the block along the line of the drill holes and the resulting blocks removed by mechanical means. As technology advances, these traditional methods are being replaced by the use of large mechanical equipment including wire saws and diamond tipped circular saws.

14.2.2.3 Uses and agents of deterioration

Stone is used in many situations including load-bearing structures, arch bridges, cladding, paving, staircases and worktops.

The best use of stone requires a good understanding by architects and suppliers of the environment in which the material is to be used and the various agents which may bring about the decay or deterioration of the stone. Three factors are important:

- The weathering agents (frost, soluble salts, acid deposition, moisture and temperatures cycles).
- The chemical make-up of the stone.
- The physical make-up of the stone.

14.2.2.4 Maintenance

Costly repair and possible replacement can be avoided by adequate and regular attention to maintenance. A prerequisite to cleaning should be a detailed survey to define the extent and type of soiling present. This may include graffiti, urban grime, rust staining, salt damage, biological growths and algae. The main types of cleaning techniques are water jetting, laser treatment, use of chemical washes, or, *in extremis*, paint-removers, grit blasting or grinding. On major projects, it is essential to set up trial areas of treatment in order to gain knowledge of the type of soiling and, by trial and error, to establish the most appropriate methods to use.

14.3 Metals

14.3.1 Cast and wrought iron

14.3.1.1 Cast iron

Cast iron typically contains up to 2.5% carbon. This has the effect of lowering the melting point below that of pure iron (normally about 1535°C). This reduction is beneficial as it increases the facility to cast the material into useful shapes of high compressive strength. However, the material has low tensile strength by comparison with structural steel and exhibits brittle failure. Cast iron was widely used in construction in the nineteenth century but has largely been replaced by steel.

It is generally recognised that there are three types of cast iron:

- **Historic grey.** Extensively used structurally between from about 1780 to 1880 for columns, beams, arches, brackets and other artefacts often moulded into attractive architectural shapes. This material is typically *grey cast iron* and is of particular interest to engineers and other professionals involved in refurbishment and repair.
- **Modern grey.** This is similar to *historic grey* but produced subject to the requirements of modern materials standards. It is, however, only used infrequently where it is necessary to structurally replicate existing repair or refurbishment work. Its robustness makes it suitable for use in drainage applications where rigidity is more important than predictable structural performance.
- **Ductile** (*spheroidal graphite*, colloquially known as *s.g. iron*). This is a high grade material almost akin to structural steel but with the advantage that it can be cast rather than rolled or forged.

14.3.1.2 Wrought iron

Rarely, if ever, manufactured today, *wrought iron* was extensively used in many buildings constructed during the nineteenth century. It is a low carbon iron, malleable and can be hammered into shape thus making it ideal for the production of ornamental ironwork such as fencing, gates, balustrades, locks, tie-rods and a variety of nails, screws and security catches. This material was the stock-in-trade for the village blacksmith. Henry Cort (1740–1800) was the mastermind behind the original production with the invention of the *puddling* furnace. However, the material is more ductile and stronger in tension than cast iron and was extensively used for the production (by hot rolling) of structural sections such as tees, angles, plates and I-beams of modest size. Some wrought iron sections may exhibit an imprint of the maker's name and size of section. The material is still of interest to those involved in refurbishment who may be required to produce articles to architecturally match those of earlier origin.

14.4 Steel

14.4.1 Introduction

Steel is essentially an alloy of iron and carbon. Depending on the required performance the carbon content will not normally exceed 1.7%. In addition, the performance can be adjusted by the addition of quantities of manganese, silicon, chromium and nickel. Steels containing more than 11–12% chromium are classed as stainless steel (available as Martenistic (not widely used in construction), Ferritic and Austenitic types). For steel required for welding purposes, the carbon content is usually restricted to 0.54% carbon equivalent. Despite exacting manufacturing processes small residues of impurities such as sulphur, phosphorus, copper, nickel and tin may still persist. These may have been present in the original pig iron and are difficult to remove.

Many types of steel are produced and it is interesting to note that E. H. Salmon in his 1930 book *Materials and Structures* listed 30 different grades of the material. The *Smithells Metals Reference Book* (Gale and Totemeier, 2003) provides a comprehensive listing of almost all available steels.

Steel superseded cast and wrought iron towards the end of the nineteenth century. In 1877 the Board of Trade (BoT) approved the use of steel for bridges and Dorman Long rolled the first joist section in 1885. It is thought that one of the first steel-framed buildings in the UK was the Ritz Hotel in Piccadilly London. Strangely, this was also the first known British use of steel sections in metric units as they were of German origin. At that time, the traditional way of connecting steel sections was by the use of cleats and rivets. A helpful book giving guidance on the early use of structural steel is the *Historical Structural Steelwork Handbook* by W. Bates (1984).

One great advantage of steel is that it is easily recycled. Corus and other industry leaders claim that almost 100% of steel is recycled (Corus Group, 2004b). The success of this from the viewpoint of quality is dependent on the correct identification and selection of the material to be recycled. Good quality assurance (QA) systems should ensure that impurities (tramp metal) are kept to a level consistent with the required quality of the recycled material.

The essential qualities for steel are that it should be:

- available in large quantities at acceptable cost;
- suitable to be fashioned into suitable sectional shapes;
- of adequate strength, toughness, durability and ductility;
- suitable for joining by welding or other devices;
- suitable for recycling.

Structural steel is available in many grades with yield strengths varying from 185 N/mm² (for Grade S185) to 360 N/mm² (for Grade E360) with a Young's modulus of 210 000 N/mm². A full explanation of these grades and their relevant mechanical properties may be found in European structural steel standard EN10025:2004. Further information on weathering steel is to be found in Corus Group (2004a).

14.4.2 Manufacture

From about 1856, steel was produced using the Bessemer process. Bessemer takes its name from Sir Henry Bessemer (1813–98). In this system, molten pig iron is loaded into a tilting furnace (known as a Bessemer converter) at about 1250°C. Air is blown into the converter from the base and spiegel (a pig iron containing a *high* content of manganese and carbon) is added. The lining of the converter acts to remove impurities to form a slag. In a final operation, the furnace is tilted to drain off the molten steel.

The Bessemer process has been largely superseded by the basic oxygen system (BOS). This is a two-stage process responsible for most of the steel currently being produced. This is an incremental process in which usable iron is first produced some of which is reprocessed into steel. The basic feedstock is iron ore and up to 25% scrap steel. Liquid iron is produced by heating the ore with coke and lime in a blast furnace. Some of this brittle material is then reprocessed to produce steel by removing some of the carbon by blowing oxygen through the metal in a convertor.

The electric arc furnace method (EAF) uses, as its feedstock, mainly scrap iron and steel. For the more critical grades of steel it is customary to use palletised iron. The industry recognises that a system that uses largely scrap material may be subject to injurious tramp material (copper, nickel and tin) so, as with the BOS system, selection of the scrap material must be consistent with the required quality of the finished product. EAF accounts for perhaps 30% of the total output.

14.4.3 Corrosion and other potential defects

14.4.3.1 Atmospheric corrosion

When exposed to moisture and oxygen, steel, being a ferrous material (containing iron) converts to hydrated iron oxide in the form of rust. This is an electrochemical process and must be countered by protecting the steel with paint or other coating. Prior to treatment it is essential to adequately prepare the metal by blasting, wire brushing or other means to remove mill scale. Red lead paints used in the past to protect the steel have been largely replaced by more sophisticated materials such as alkyds, chemical resistant or bituminous paints, epoxides or urethanes. Metallic coatings such as hot dip galvanising, electroplating or sheradising are also available for particular applications.

In certain environments that are free from chloride contamination it is possible to use weathering steels as an alternative to more conventional steels and which do not require anti-corrosion coatings. The specific, alloying elements produce a stable oxide layer that adheres to base metal and is less porous than the rust on other steels. Chloride contamination might occur from seawater spray, salt fogs or salts used to clear snow and ice from roads in winter. Corus recommend that weathering steel should *not* be used within 2 km of a coastline. A striking use of weathering steel is the *Angel of the North* sculpture near Gateshead in the UK. A limited number of structures have been constructed using this material and, in the dry clean atmosphere of Tenerife, a footbridge has been built.

Resistance to atmospheric corrosion may be enhanced by coating the steel with resistant coatings or galvanising. If galvanising is used, reference to the section on liquid metal assisted cracking (LMAC – see section 14.4.3.3 below) should be made.

14.4.3.2 Bi-metallic corrosion

The corrosion of metals is basically electrochemical in nature and takes place in the presence of an electrolyte (a solution containing ions). Although pure water is not a good electrolyte it is, in practice, often polluted by small amounts of salts, acids or alkalis which considerably increase the number of ions.

When two metals of differing galvanic potential are in close contact with an electrolyte, an electric current passes between

Magnesium	Anodic
Aluminium	↑
Zinc	
Chromium	
Iron (Fe^{2+})	
Nickel	
Tin	
Lead	
Iron (Fe^{3+})	
Hydrogen reference	
Copper	
Silver	↓
Gold	Cathodic

Figure 14.4 Electrochemical series for pure metals

them and the metal with the lower potential value (the anode) corrodes. Some metals, for example, copper and nickel, therefore accelerate the corrosion of steel; others such as zinc corrode preferentially and actually protect the steel. The rate of bi-metallic corrosion increases with the relative separation of the two metals in the electrochemical series (see **Figure 14.4**). It also depends on the nature of the electrolyte and the contact area.

Bi-metallic corrosion can be most severe in immersed or buried structures. In less aggressive environments where, for example, stainless steel brick support angles are attached to mild steel structural sections no special precautions are required. The problem can be avoided altogether by isolating the two adjacent metals with gaskets of neoprene or similar material.

14.4.3.3 Liquid metal assisted cracking (LMAC)

LMAC is a rare phenomenon that can take place when steelwork is galvanised to provide protection from corrosion. Certain solid metals with other liquid metals can give rise to a reaction which may affect the parent solid material. This reaction is termed liquid metal embitterment (LME) and may lead to cracking of the steel. For example, when structural steel is stressed and temporarily in contact with liquid zinc in the galvanising process then LME/LMAC may occur. More research will identify more clearly the critical factors but stress level, material susceptibility and the presence of a liquid metal are thought to be the main elements of the problem.

14.4.4 Performance in fire

Hot finished carbon steel begins to lose strength at temperatures above 300°C and then reduces in strength at a steady rate up to 800°C. The small residual strength then reduces gradually until meltdown at around 1500°C. For cold worked steels there is a more rapid decrease in strength after 300°C. The thermal properties of steel at elevated temperatures are found to be dependent on temperature rather than stress level and rate of heating.

Until recent times it has been the practice to allocate fire resistance ratings to individual elements of construction, these ratings being relevant to the building or structure. Thus for a typical office block a rating of two hours' fire resistance might be deemed suitable. All structural elements would then be treated to meet that criterion. Steel members would be cased in concrete or other materials such as asbestos (now banned) or vermiculite boarding. As an alternative, a coating of intumescent paint of the appropriate thickness might be applied to steel members. However, a more modern approach termed *fire engineering* takes a more holistic view of a structure taking account, for example, of large-scale fire tests carried out on a 12-storey steel-framed building at the BRE testing facilities at Cardington. These tests proved that a steel-framed building designed for a specific resistance did not necessarily immediately collapse after the expiry of that time. This is due to the structural continuity and inherent robustness of the frame. It is also apparent that a heavy, massive steel section will heat up more slowly than a light slender section. Modern fire engineering methods permit the calculation of the fire resistance of uncased steel.

Under the leadership of Professor Colin Bailey of Manchester University a one-stop facility for dealing with Fire in Structures is available at www.mace.manchester.ac.uk.

14.4.5 Design

Structural design techniques are dealt with elsewhere in this book. However, it is relevant to point out that two developments have radically changed the way the material is designed. These are:

- The change from permissible stress to limit state methods.
- The advent of Eurocodes; in particular the issue of BS EN1993 Eurocode 3: Steel.

14.4.6 Structural sections available to designers

The range of sections available to design engineers is considerable and includes:

- Universal beams and columns
- Universal bearing piles
- Sheet piles
- Joists
- Parallel flange channels
- Angles (equal and unequal)
- Z sections
- Slimdeck beams
- Tees
- Structural hollow sections (square, rectangular and circular)
- Pressed metal sections (available from specialist suppliers)
- Structural section in accordance with European, Japanese wide flange and ASTM specifications

- Special corner and junction pieces
- Plates
- Bolts and other fixing devices.

Note: Since about 2007, following alignment with European Standards, Corus have produced a new range of CE marked sections (see **Tables 14.1**, **14.2** and **14.3**) using UK rather than UB nomenclature. The earlier 43 and 50 grades have largely been replaced by 275 and 335 grades. For engineers working on refurbishment and/or repair projects, it may be necessary to refer to old section books for information on section profiles. In this connection the Institution of Structural Engineers Library has access to a wide range of old section books. Readers are also directed to Bates (1984).

- Reinforcement for concrete (including pre-stressing wires and cables)
  - Carbon steel bars for reinforcement of concrete (BS 4449:2005)
  - Welded steel fabric for the reinforcement of concrete (BS 4483: 2005)
  - Pre-stressing steel wire and strand (BS 5896:1980) and bars (BS 4486:1988) for the reinforcement of concrete
  - Cold worked steel.

Tata Steel Advance® sections		Old designation system	
UKB	UK Beam	UB	Universal Beam
UKC	UK Column	UB	Universal Column
UKPFC	UK Parallel Flange Channel	PFC	Parallel Flange Channel
UKA	UK Angle	RSA	Rolled Steel Angle
UKBP	UK Bearing Pile	UBP	Universal Bearing Pile
UKT	UK Tee		

Table 14.1 Section designation system (Corus 2007) (Reproduced courtesy of Tata Steel)

In addition to the above there is developing experience of casting high grade steel to form spherical sections for the nodes of space frames and other applications.

14.5 Aluminium

Aluminium (Al) is the most abundant metallic element in the earth's crust. It was isolated initially by H. C. Oersted (1777–1851) in 1826 and first produced commercially in 1886. It is a component of a number of elements; however, in most hosts the aluminium content is too small for economic extraction. The exception is bauxite which contains hydrated alumina together with oxides such as iron, silicone and titanium. These deposits are usually near the surface and can be mined by opencast quarrying. Deposits are found in many parts around the world, the principal areas being Australia, Brazil, China, Guinea, Jamaica and India. In 2006, more than 177 Mt of bauxite

Section type	Advance® designation	Dimensions	Tolerances
UK Beam	UKB	BS4-1:2005	BS EN10034: 1993
UK Column	UKC		
UK Bearing Pile	UKBP		
UK Parallel Flange Channel	UKPFC	BS4-1:2005	BS EN10279: 1200
UK Angle	UKA	BS EN10056: 1999	BS EN10056-2: 1993
UK Tee (cut from Universal Beams and Universal Columns)	UKT	BS4-1:2005	
ASB (Asymmetric Beam) Slimdek® Beam	ASB		Generally BS EN10034:1993

Table 14.2 Standards applicable to Advance® sections from Tata Steel (Corus 2007) (Reproduced courtesy of Tata Steel)

Advance® sections	EN10025:Part 2:2004					BS 4360:1990
Grade	Grade	Yield (R_mH^a) min	Tensile ($R_m{}^a$)	Charpy v-notch longitudinal		Grade
		Strength at t = 16 mm (N/mm²)		Temp(°C)	Energy(J) t = 16 mm	
Advance275JR	S275JR	275	410/560	20	27	43B
Advance275JO	S275JO	275	410/560	0	27	43C
Advance275J2	S275J2	275	410/560	–20	27	43D
Advance355JR	S355JR	355	470/630	20	27	50B
Advance355JO	S355JO	355	470/630	0	27	50C
Advance355J2	S355J2	355	470/630	–20	27	50D
Advance355K2	S355K2	355	470/630	–20	40	50DD

Example – EN10025:Part 2:2004 – S275JR becomes Advance 275JR

Table 14.3 Comparison of grades for Advance® sections, EN10025:Part 2:2004 and BS 4360:1990 (Reproduced courtesy of Tata Steel) was extracted, 83% of which became a source of aluminium. Aluminium is extracted from bauxite by the Hall–Héroult electrolysis process. Four tonnes of bauxite makes two tonnes of alumina which, in turn, produces one tonne of aluminium

Aluminium can be alloyed with traces of other elements such as magnesium, manganese, chromium and silicon. A strict definition of an aluminium alloy is one that contains at least 99% by mass of aluminium and stays within the following limits:

- A total content of iron and silicon not greater than 1%.
- A content of any other element not greater than 0.10% except for copper which may have a content up to 0.20% provided that neither the chromium nor the manganese content exceeds 0.05%.

Aluminium alloys can be classified as:

- Heat treatable (can be strengthened by thermal treatment).
- Non-heat treatable (cannot be strengthened by thermal treatment).
- Castable (by sand, die and/or centrifugal casting methods).

Aluminium alloys are corrosion-resistant in many environments due to the inert film of aluminium oxide which forms on its surface. Certain types are weldable using both MIG (metal inert gas) and TIG (tungsten inert gas) systems. For structures at modest stress level and under conditions of tight quality assurance/control procedure it is also possible to use adhesives to joint structural members. Alloys are defined by a complex alpha-numeric coding system (see below) which defines properties such as strength, temper, weldability and others. Decorative effects can be achieved by anodising and other techniques. Examples of use include:

- Lightweight prefabricated buildings
- Space frames
- Motorway gantries
- Roofing members
- Structural members in aircraft
- Offshore heli-decks
- Pipework and ducting
- Scaffolding
- Curtain walling and other types of cladding
- Lighting columns
- Transport vehicles (where weight reduction can lead to reduced fuel demand).

Aluminium may be subjected to the following types of corrosion:

- Galvanic (bi-metallic corrosion)
- Pitting
- Intergranular
- Exfoliation
- Stress cracking.

In fire, aluminium alloys will melt at around 550°C to 650°C and start to lose strength at temperatures in excess of 100°C.

Aluminium alloys can be repeatedly recycled. Each operation only takes about 5% of the energy used for the original manufacture of new alloys. Good quality scrap is recycled for the production of extruded and rolled products. Depending on the type of scrap material, recycling rates in the UK can be as high as 98%. It is reported that the recycling rate for aluminium cans in Japan and Brazil is over 90%.

Aluminium and aluminium alloys are classified using an alpha-numeric system. To give but one example BS EN573-1 gives the European designations as follows.

EN AW-5154A: EN shows it is a European designation listed in a European Code. EN is followed by a blank space. A represents aluminium and W represents a wrought product. After the W the hyphen is followed by the international designation consisting of four digits representing the chemical composition and, if required, a letter identifying a national variation: this designation is attributed by the Aluminium Association via an international registration procedure.

The following are some typical ranges of property values:

- Tensile strength 55 N/mm^2 to 580 N/mm^2
- Proof stress (0.2%) 60 N/mm^2 to 520 N/mm^2
- Modulus of elasticity 69 000 N/mm^2 to 80 000 N/mm^2
- Coefficient of linear expansion 16×10^{-6} to 24×10^{-6} per °C.

14.6 Concrete

Concrete is not a new material: it is claimed that the first example was its use in the floor of a hut in Yugoslavia in 5600 BC. Concrete is strong in compression but weak in tension. Until the late nineteenth century it was, for the most part, unreinforced and performed well in arched structures such as short span bridges. At the end of the century under the influence of Coignet, Hennebique and others methods of reinforcing the material were introduced and *ferroconcrete* was born.

In building structures, *in situ* and/or precast concrete may be used. In precast structures care must be taken with the detailing of joints and connections to achieve the required stability of the whole structure. In water retaining structures, such as reservoirs, watertightness may be achieved by best practice design, detailing and construction.

Before, and immediately after, the Second World War a cubic yard of concrete might contain Portland cement, all-in ballast, or a mixture of natural coarse and fine aggregates (usually crushed rock) and sufficient water for adequate hydration of the cement; reinforcement would probably be plain round mild steel bars. The concrete would most probably have been volume batched. Compaction might have been achieved by hand tamping or early types of surface or poker vibrators.

Curing, if carried out, would have been by damp sand or hessian. Mixes of 1:2:4 achieving an allowable strength in compression of 3000 psi were the order of the day.

Today a cubic metre of concrete might contain a wide variety of cements of which there are now 90 types manufactured by British Cement Association (BCA) companies. Cements may be blended with pulverised fuel ash (PFA) and/or ground granulated blast furnace slag (GGBS) and/or microsilica. There should be sufficient water for hydration. The reinforcement might be a mix of mild steel, high tensile with plain or deformed cross-sections. The reinforcement might also be stainless steel or possibly epoxy-coated or galvanised. Alternatively, reinforcement may be by way of carefully controlled doses of steel and/or polymer fibres evenly distributed throughout the mix. Various additives to improve workability and/or to accelerate/retard strength gain might be present and the mix might be air-entrained (too much air entrainment might lower the strength of the concrete). Most mix materials would be weigh-batched. It is also likely that the concrete would be delivered to the site ready-mixed and possibly pumped into position. Compaction could be achieved by sophisticated vibration techniques. Curing would most likely be by use of a sprayed chemical membrane. Characteristic strengths can vary between 2 N/mm^2 (for no-fines concrete) to well in excess of 100 N/mm^2. The increased complexity brings with it many benefits but also more chance of error and loss of long-term durability.

Concrete is specified in grades to BS 8500 and BS EN206. Standard grades vary from C25/30 to C50/60 where the numeric symbol is the cylinder strength and cube strength respectively in N/mm^2. Nominal cover to reinforcement usually varies between 25 mm and 60 mm. Recommended values are available for C40/50 concrete made with ordinary Portland cement (OPC) as being satisfactory for a 50 year life. Steel bar reinforcement is usually high tensile deformed bar to BS 4449:2005 with a characteristic yield strength of 500 N/mm^2. Bars are classified H6 to H40 being 28 mm^2 to 1257 mm^2 in cross-sectional area respectively. For slab reinforcement high tensile fabric (to BS 4483:2005) is also readily available in sheet or roll format.

Dr George Somerville in his 1986 IStructE award-winning paper has argued that the four essentials for good reinforced concrete are special attention to the four Cs: Constituents, Compaction, Cover and Curing.

A range of special cements is available; these include sulphate resisting cement (SCPC) in which the tri-calcium aluminate content is controlled to a low level. However, it has been shown that the resistance to the thaumasite form of sulphate attack may not be sufficiently controlled by SCPC in cool ground conditions. For further advice, readers are referred to DETR (1999).

In some countries, there is a variety of hydraulic cements available for other special purposes such as those used to offset cracking due to shrinkage, those used for work in high temperatures and those that are finely ground in which the constituents are selected to react early with water, and those for specialist use in rendering, plastering and masonry work.

In 1973, the roof of the assembly hall at the Camden School for Girls collapsed. In 1974, a similar collapse occurred over the swimming pool at Sir John Cass's Foundation & Redcoat School in Stepney, East London. Investigations revealed that the use of high alumina cement (HAC) in the precast prestressed concrete beams to these roof structures was the principal cause of collapse. Concrete made using HAC may be subject to conversion causing a large loss of strength. The use of this cement in concrete was of considerable advantage to manufacturers because it gained high early strength thus enabling formwork to be struck early and immediately reused. As a result of these disasters the use of HAC for structural purposes is now banned under the Building Regulations.

Concrete, including reinforced concrete, subjected to atmospheric conditions also incurs carbonation. When carbon dioxide in the air combines with rainwater it forms carbonic acid. The alkalinity of the protective concrete of cover to reinforcing steel is reduced by the carbonic acid so that water and oxygen attack and corrode the steel. This neutralisation is known as carbonation. The rate at which it proceeds from the surface depends on a number of factors such as porosity and type of cement. One authority has quoted that carbonation proceeds at a rate of 5–10 mm every 10 years.

Other potential defects include alkali silica reaction (ASR). This reaction requires the presence of

- a high alkali cement,
- a reactive aggregate and
- moisture.

This problem has been identified in at least 50 countries around the world.

When damaging ASR is present, the concrete cracks (often with an Isle of Man symbolic three-legged appearance) and, in the most severe cases, will require demolition and replacement of the structure. In less severe cases it may be possible to lengthen the life of the structure by removing the source of the water. Such structures should then be subjected to regular monitoring to check on the efficacy of the remedial measures.

It is important that those involved in repair and refurbishment of structures recognise the many changes that have taken place in the development of concrete and the need to understand the contemporary environment in which the structure under consideration was designed and constructed.

14.6.1 Performance in fire

Well designed and constructed reinforced concrete has good inherent resistance to fire. BS 8110-1 states that

> A structure or element required to have fire resistance should be designed to possess an appropriate degree of resistance to flame penetration, heat transmission and collapse.

BS 8110-2 gives recommendations for cover to reinforcement based on element shape and mix constituents. It also allows benefit for additional protection such as gypsum plaster. Attention is also drawn to the fact that the fire resistance of the whole structure may be greater than that ascribed to individual elements. Reinforcement of cold worked steel shows a rapid decrease in strength after 300°C. In well designed and constructed concrete this should be adequately protected from fire by the cover provided to the reinforcement. For those involved in repair and/or refurbishment, reference to **Table 14.4** will give a guide to standards current before the 1990s and possibly some beyond that date. For a detailed account of structural fire engineering methods see Chapter 12: *Structural fire engineering design*.

The approach to fire resistance has changed radically in recent times and reference to www.structuralfiresafety.org provides a one stop appraisal of Fire Protection Engineering.

14.7 Timber

14.7.1 Introduction

Timber is ubiquitous; it is sustainable, it has a strength/weight ratio that is better than mild steel when loaded in its strong direction, and, in forests, it is beneficial to the climate as it absorbs damaging carbon dioxide. The original limitation of usable length related to the size of tree has largely disappeared with the development of high strength and durable adhesives. These permit the manufacture of long span structural elements (such as Glulam beams) and also a wide range of boards such as:

- Plywoods, including weather-resistant marine ply, blockboard and laminboard.
- Particle boards including chipboard and cement particle-board.
- Fibre building boards (including MDF – medium density fibreboard).

The UK only produces about 20% of the timber used in the country so much of its supply has to be imported from elsewhere in Europe and beyond.

Wood is a cellular material and anisotropic (different properties in different directions). In addition to natural defects such as knots, shakes and wane, timber may be adversely affected by rot. This occurs in two manifestations: dry rot (caused by the fungus *Serpula lacrymans* and which is more prevalent in

		Nature of construction and materials		Minimum dimensions (mm), excluding any finish, for a fire resistance of					
				½ h	1 h	1½ h	2 h	3 h	4 h
Slabs: ribbed open soffit	1	Reinforced concrete (simply supported)							
		(a) Normal weight concrete	thickness	70	90	105	115	135	150
			width	75	90	110	125	150	175
			cover	15	25	35	45	55	65
		(b) Lightweight concrete	thickness	70	85	95	100	115	130
			width	60	75	85	100	125	150
			cover	15	25	30	35	45	55
	2	Reinforced concrete (continuous)							
		(a) Normal weight concrete	thickness	70	90	105	115	135	150
			width	75	80	90	110	125	150
			cover	15	20	25	35	45	55
		(b) Lightweight concrete	thickness	70	85	95	100	115	130
			width	70	75	80	90	100	125
			cover	15	20	25	30	35	45
Walls	1	Less than 0.4% steel Normal-weight aggregate	thickness	150	150	175	200	–	–
	2	1% steel Normal weight aggregate (concrete density 2400 kg/m^3)	thickness	100	120	140	160	200	240
			cover	25	25	25	25	25	25
	3	More than 1% steel Normal weight aggregate (concrete density 2400 kg/m^3)	thickness	75	75	100	100	150	180
			cover	15	15	20	20	25	25
	4	Lightweight aggregate (concrete density 1200 kg/m^3) (Note: intermediate densities may be interpolated)	thickness	100	100	115	130	160	190
			cover	10	20	20	25	25	25

Table 14.4 Fire resistance of reinforced concrete (IStructE, 1991) © The Institution of Structural Engineers, 1991

softwoods than hardwoods) and wet rot (caused by a fungus other than *Serpula lacrymans* such as *Coniophora puteana* and which characteristically attacks wet timbers). The performance of timber is closely related to its moisture content which should be kept below 25% for good performance and at the lower level of 20% for structural applications.

Timber may also be attacked by insects although in temperate climates insect attack will not normally occur. However, one exception concerns the House Long Horn Beetle (*Hylotrupes bajulus*) which is largely confined to some parts of southern England as listed in the UK Building Regulations. In these areas the use of preservative treatments in roof timbers is mandatory.

Some timbers, such as oak and Western cedars, are acidic and, if wet, may corrode embedded ferrous metal fasteners. In such cases, it is prudent to use stainless steel fasteners. Timber is susceptible to certain natural defects such as knots, shakes, wanes and splits so care must be taken in the selection of timber to match the intended purpose.

Traditionally, timber has been classified into hardwoods and softwoods. This is a botanical distinction unrelated to the density of the material. The following are a few examples and their common usage in construction.

- Hardwoods
  - Ash (European) – interior joinery
  - Elm (European) – furniture, rubbing strips
  - Birch (European) – plywood, flooring
  - Greenheart (Guyana) – heavy construction, piling, lock gates, etc. Classed as very durable and extremely resistant to attack particularly in tidal zones
  - Cedar (Central/South American) – cabinet work and interior joinery
  - Maple (N. America) – flooring, furniture
  - Oak – structural applications, flooring, fencing, interior and external joinery
  - Balsa
- Softwoods
  - Parana pine (S. America) – plywood
  - Pitch Pine (S. America) – heavy construction, interior and exterior joinery
  - Scots Pine (Scotland) – construction, carpentry
  - Spruce (N. America) – construction, carpentry
  - European Whitewood (Scandinavia, Russia) – construction, flooring
  - Yew (Europe) – furniture, interior joinery

Stress grading is carried out in one of two ways:

- Visually by experienced operators working to standards laid down by the NLGA (National Lumber Grading Authority in Canada) or NGRDL (National Grading Rule Dimension Lumber in the USA).
- By machine.

Structural timber is classified into a number of grades: C14 to C50 for softwoods and D30 to D70 for hardwoods. In these grades the numeral refers to the bending strength in N/mm^2. Strength properties are usually assessed at 20°C temperature and 65% relative humidity

Timber is anisotropic, that is to say the strength varies with direction. For example, for a class C24 softwood the compression strength parallel to the grain might be 21 N/mm^2 but only 2.5 N/mm^2 perpendicular to the grain. Young's modulus varies in similar fashion from 11 N/mm^2 to 0.37 N/mm^2.

14.7.2 Performance in fire

Although timber is classified as a combustible material, a well designed timber structure can perform well in fire. However, in many structures timber members will be protected by fire resistant material such as plasterboard. One layer of plasterboard will provide half an hour fire resistance and it is not uncommon to find more critical parts of structures protected by two thicknesses of plasterboard. Heavy timber construction has good inherent fire resistance due to the charring effect. When heavy timber members are exposed to fire, the temperature of the fire-exposed surface of the members is close to the fire temperature. When the outer layer of the wood reaches about 360°C the wood ignites and burns rapidly. The burned wood becomes a layer of char which loses all its strength but retains a role as an insulating layer preventing an excessive temperature rise in the core. It is interesting to note that examples exist of heavy structural timbers coated with intumescent paint in order to increase the fire resistance of the members.

14.8 Polymers

14.8.1 Introduction

Polymers include the following materials:

- polyethylene
- polypropylene
- polycarbonate
- acrylics
- polystyrene
- PTFE (polytetrafluoroethylene)
- thermosets
- elastomers and rubbers
- polymer dispersions
- silicones, silanes and siloxanes

Polymers are found in many applications. It is beyond the scope of this book to provide a detailed analysis of all those that are to be found in construction. However, a guide to those most in common use is provided below to give engineers an introduction to the subject. Fuller coverage may be found in the references at the end of the chapter. Plastics may be broadly classified under two headings.

- *Thermoplastics* – materials that can repeatedly be softened by heating and hardened again on cooling.
- *Thermosetting* – materials that are initially soft but change irreversibly to a hard rigid form on heating.

14.8.1.1 Vinyls

The full scientific name for a vinyl is *polyvinyl chloride*. The correct nomenclature for a flexible vinyl is PVC-P and that for a rigid vinyl is PVC-U (often referred to as UPVC). PVC is a polymer of vinyl chloride and contains 57% chloride. It is generally derived from oils and salt by electrolysis methodology in Europe but, in other countries such as China, other methods are used.

The most common uses of PVC in the UK are rigid formulations for door and window frames and flexible types for pipework where it is claimed to provide for half the European market. Other uses include for cable covers and flooring.

The PVC industry makes many claims for the material including:

- Excellent strength to weight ratio.
- High tensile strength and resistance to pressure.
- Good flexibility, durability, creep characteristics, resistance to abrasion and bacterial attack.
- Depending on formulation – low flammability.
- Ease of jointing – particularly in pipework.
- Ease of recycling without loss of essential qualities.
- Life expectancy assessed to be in excess of 30 years.

14.8.2 Polypropylene

Polypropylene is a stiff, chemically resistant translucent thermoplastic material. It is widely used for domestic packing, containers, automotive parts and rope fibres. Similar to polyethylene it has a higher softening point and a good resistance to cracking. In civil engineering it is extensively to be found in pipes and pipe fittings, drainage access chambers, membranes, damp-proof courses, formwork for concrete, fibre reinforcement and storage tanks.

The material had its origins in Natta, Spain in around 1954 when polypropylene was first produced from polypropylene gas. Commercial production began in 1957. Industrial production of butane-1 commenced in Germany in 1964 closely followed by manufacture in America, Italy, Holland and Japan.

Polypropylene will burn in fire, produces soot and is injurious to humans due to its adherence to the skin. Continued sustainability is closely related to the availability of oil. Recycling is possible but many applications require production from virgin materials.

14.8.2.1 Polycarbonates

Polycarbonates (PCs) are part of the family of thermoplastics. They were discovered by Dr D. Fox (General Electric) and Dr H. Schnell (Bayer) in about 1955. Both companies applied for patents and commercial production began in 1960. PCs are flame retardant, impact modified, high melt strength and glass fibre-reinforced products. World annual production is currently 2 Mt. Blended with ABS (acrylonitrile-butadiene-styrene) and polyester its physical properties can be enhanced. Rubber-modified polycarbonate also has enhanced impact resistance. Polycarbonate is basically a slow burning plastic, but flame resistant grades are available. In general, with some other plastics, polycarbonates can be recycled by grinding and pelletisation for reuse.

Component manufacture is mainly by extrusion, injection moulding, vacuum forming or blow moulding. In its foam formulation, it appears in sandwich form. PC sheets are used extensively for roof lights and domes, shelters, car ports, road barriers, greenhouses and covered walkways. High specification material may be used in bullet-resistant laminates which absorb impact energy and avoid dangerous spalling. Trade names for the material include Makrolon, Lexan, Xantar Panlite and Zelux.

Its durability is somewhat limited because it can be easily scratched although removal polish can deal with light scratching. It also has a tendency to yellow when exposed to sunlight for long period, although there are specialist grades available which can limit this problem.

14.8.2.2 Acrylics

Acrylic plastics are a form of thermoplastic polymers. They include polymethyl mettacryolate (PMMA); polyacrylonitrite (PAN) and cyanoacrylates (CA). CA was first produced in 1880 by Swiss chemist Georg Kahlbawn and then commercially by Rohm & Hass in 1927. PMMA was first manufactured by ICI in 1934 under the trade name Perspex. DuPont followed, using the trade name Lucite. It is currently available in sheet or granular form. PAN is frequently used in textile fibres and is the forerunner of carbon fibre. CAs are the feedstock of superglues and PMMAs form the basis of acrylic paints. Acrylic modified concretes find applications in concrete repair materials. In addition to Perspex and Lucite trade names for PMMA include Plexiglass, Acrylite and Polycast amongst others.

PMMA is produced in large quantities, exceeding 3 Mt in 2005. Acrylics are usually produced by extrusion or casting techniques. Clear cast acrylic sheet is available in a wide range of thicknesses from 2 mm to 50 mm, in sheet sizes up to 3050 mm × 2030 mm. Extruded material (cheaper than cast) is manufactured up to 2050 mm in width and, subject to transportation restrictions, in lengths exceeding 3050 mm. A wide range of colour is available and many grades of transparency and translucency.

PMMA burns and, in so doing, generates CO_2 and CO. The sustainability of acrylics generally will depend, in the short term, on the ability of the oil industry to continue to supply the raw materials and, in the long term, for scientific development to open up new sources of supply – e.g. agriculture has been suggested as a possible source.

14.8.2.3 Polystyrene

Polystyrene is a clear, non-crystalline, brittle plastic material. Surprisingly it was first made 135 years ago but was not made commercially until 1936. By that time, 800 t was produced in Germany rising to 5000 t by 1942. Polystyrene is produced by both an expansion and an extrusion process and is usually referred to by one of the following abbreviations.

- **PS** – a foam.
- **EPS** – a foam moulded into blocks, boards and other shapes sometimes referred to as *beadboard*.
- **XPS** – an extruded foam manufactured into boards.

Polystyrene has good compressive strength. This allows use under floors and even in road construction. Although its tensile strength is modest it is usually sufficient to withstand damage during transportation. It is widely used in concrete formwork and can easily be removed after use by melting. In recent times it has found an application under suspended concrete floors to dwellings in areas subject to swelling and shrinkage of sensitive clays.

As insulation in cavity wall construction there are two systems:

- as panels fixed to the internal leaf;
- as foam sprayed into the cavity (this is usually a retrofit operation).

Polystyrene is combustible and also toxic to the extent of breaking down into CO_2 and CO during the combustion process.

14.8.3 Polytetrafluoroethylene (PTFE)

Well known in domestic applications as a non-stick material used in saucepans and other kitchenware, PTFE has a number of uses in engineering. These make use of low friction properties when associated with other materials. PTFE is produced by the polymerisation of the monomer tetrafluoroethelene. It was discovered in 1938 by Dr Roy Plunkett, a chemist working for DuPont. In working on refrigerant gases he noticed that a frozen compressed sample of PTFE had spontaneously polymerised into a white waxy solid. The trade name Teflon was registered heralding the start of commercial production.

PTFE has good dialectic properties and is an excellent electrical insulator, is not wetted by water and is non-absorbent. Its tensile strength is relatively low but has good impact resistance even at sub-zero temperatures. Some characteristics can be modified by the addition of fillers such as glass or carbon fibres and metals. The coefficient of friction is between 0.4 and 0.09 making it ideal material for the offshore industry when skidding large assembly oil rigs from barges on to sub-sea foundations. In addition, PTFE is used in bearings (particularly for bridges), pipework, low friction industrial components and coated tensile fabrics.

PTFE is an expensive material so it is important to keep wastage to a minimum. Waste in the manufacturing process is usually cleaned and ground down into powder for reuse. It is combustible and at high temperature releases toxic chemicals. PTFE is fully recyclable.

14.8.4 Thermosetting resins

The term synthetic resin is used to describe man-made thermosetting pre-polymers. Some are solids with a low melting point and many have the viscous, sticky consistency of naturally occurring substances similar to those secreted from coniferous trees. Epoxy resins were first discovered in the late 1980s followed by commercial production by CIBA of Switzerland in the early 1950s.

The scientific terminology can be confusing since both cross-linked polymers and some pre-polymers are commonly grouped together and described as resins. Thus, one component of a two-pack epoxy resin adhesive is an epoxy resin which, on reaction with the second component (the hardener or curing agent) gives a cured adhesive that is also referred to as an epoxy resin. Those systems used in construction that contain formaldehyde are used in laminates, mouldings, adhesives, surface coatings and as binders in chipboard. These products are almost always factory produced as they require both heat and pressure. Other resins, such as furanes and polyurethanes, can be used on site where curing takes place at the point of service and at ambient temperatures.

Elevated temperatures do not cause thermosets to melt and flow but do induce softening and changes in properties such as strength and chemical resistance.

14.9 Glass

14.9.1 Introduction

Flat glass has been made through the centuries since Roman times. Methods include casting, rolling, spinning, blowing, floating and drawing. Today a wide range of glass products are available, 90% of which are based on the float process that was developed by the Pilkington brothers in the 1950s. This section focuses on the glass products derived from the float process.

14.9.2 Primary manufacture

Float glass used in the building industry is generally referred to as soda-lime-silica glass. The process involves the following stages:

1. Melting of the raw materials 72% silica sand (SiO_2); 13% sodium carbonate (NA_2 CO_3); 10% calcium carbonate (CA CO_3) and 4% calcium magnesium carbonate ($MgCa(CO_3)_2$) in a regenerative furnace at 1500°C. This can include up to 20% of recycled glass.

2. Forming the glass in the float bath, wherein a continuous ribbon of molten glass is fed from the furnace onto a long bath of molten tin. As it floats on the tin, the glass ribbon reaches an equilibrium thickness of approximately 7 mm. The glass ribbon is stretched or compressed by varying the speed of the take-out rollers and by positioning guides, thus producing thickness ranging from 2 mm to 25 mm. Crystallisation is prevented by cooling the glass rapidly from around 1000°C to 600°C.
3. Cooling the glass gradually and uniformly from 600°C to 200°C in the annealing lehr (furnace). This eliminates residual stresses and makes the glass suitable for cutting.
4. The glass is checked for optical faults in the form of small inclusions, bubbles, lack of flatness and glass inhomogeneity (BS EN572:2004; ASTM C1036:2011). The checking process is usually automated and generally involves illuminating the glass onto a perfect white surface. The glass is subsequently cut by a computerised process, after which it is batched for warehousing or processing.

The resulting annealed float glass has the following properties:

Optical properties. A predominantly transparent material. The spectral transmittance of glass ranges from 300 nm to 2500 nm. Note that this includes a proportion of UV radiation and near infrared radiation.

Mechanical properties. A brittle material whose strength is governed by the presence of surface flaws. This is discussed further in Chapter 18.

Thermal properties. The coefficient of thermal expansion of soda-lime glass is about 9×10^{-6} /°K. Glass has a relatively high thermal conductivity of approximately 1.0 W/mK.

Chemical properties. Glass is very durable; and it is inherently resistant to most aggressive substances except hydrofluoric acid and hot alkaline solutions.

Fire resistance. Glass is non-combustible, but loses all its strength around 700°C and is unlikely to withstand a temperature difference of more than 60°C without fracturing. Furthermore almost 100% heat radiant can pass through glass therefore causing combustible elements beyond the glass to ignite and/or preventing people from using the space as a safe means of exit in the event of a fire.

Acoustic performance. Glass is a poor acoustic insulator; however, this effect can be countered by using double and triple glazing with different glass thicknesses and gas filled cavities.

14.9.3 Modified primary manufacture

Three common modifications to the aforementioned basic manufacturing process are:

1. Adding metal oxides to the constituents of the melting furnace in order to produce body tinted glass. These small additions colour the glass bronze, green, blue or grey with the effect of reducing solar energy transmission by up to 30% and light transmission by up to 60% with respect to basic float glass.
2. Bombarding the glass surface in the float bath with metal ions, such as titanium and copper, in order to produce surface modified glass. Reflective coatings produced by this process may reduce solar energy transmittance by up to 60% with respect to basic float glass.
3. Rolling glass, which is generally used for making wired or profiled glass. In this process, the float bath is replaced by rollers.

14.9.4 Secondary manufacture

These methods are sometimes referred to as off-line, as they involve improvements to the float glass by processes that take place after the float glass has been produced.

Heat treated glass: The principal glass produced by this process is known as fully toughened glass or tempered glass. This involves heating the glass to around 625°C and quenching. This creates a parabolic stress profile through the thickness of the glass, where the outer surfaces are in compression and the core is in tension. This has two advantages. Firstly it increases the tensile strength of the glass as any load-induced stress must exceed the surface pre-compression before failure can occur. Secondly the glass will fail in small rounded fragments rather than the sharp shards that characterise annealed glass failure. An alternative heat treatment that involves a slower quenching rate is heat strengthened glass. This produces a lower surface pre-compression than fully toughened glass and the resulting fragmentation pattern is similar to that of annealed glass. For further information, refer to BS EN12600 (2004) and ASTM C1048 (2004).

Chemically strengthened glass: Produced by immersing the float glass into a bath of potassium salt. This induces the replacement of the sodium ions in the glass surface with the potassium ions, which have a 30% larger radius. As a result, a thin compression layer is produced on the glass surface. Commercial soda-lime glasses can be strengthened, to around 300 MPa, but the process is most effective with thin aluminosilicate glass where the level of surface compression can exceed 700 MPa.

Laminated glass: Consists of bonding two or more sheets of glass with an adhesive interlayer. One method consists of pouring a self-curing resin between sheets of glass. This process had the advantage of filling cavities created by fluctuating dimensions. The more popular process consists of using plastic interlayer films, usually polyvinyl butyral (PVB), to a thickness of 0.38, 0.76 or 1.52 mm. The translucent PVB is cut and layered between glass sheets and is transformed into a clear and strong adhesive by heating to 150°C at a pressure of 860 kPa. Other interlayers such as ionoplast or ethylene vinyl acetate (EVA) are available for specialist applications such as for blast and impact resistance and for embedding photovoltaic cells in glass. Laminated glass can incorporate several thicknesses and combinations of annealed and toughened glasses. When laminated glass is broken, the interlayer tends to prevent the fragments of broken glass from falling out and may be therefore considered a safety glazing material. There is a range of performance tests that can be performed to assess the suitability of laminated glass for a given application. These are described in Chapter 18.

Curved glass: Produced by either heating flat glass beyond its softening point or by bending the glass at ambient temperature (cold-bending). The most popular heat bending process is sag bending, wherein the glass is heated to around 700°C at which point the softened glass relaxes onto a mould. Single curvature sag-bent glass is limited to a radius of curvature of:

- 100 mm for 6 mm thick glass
- 300 mm for 10 mm thick glass
- 750 mm for 12 mm thick glass
- 1000 mm for 15 mm thick glass
- 1500 mm for 19 mm thick glass

Double curvature bending is available from specialist glass processors.

Other processes: There are several other on- and off-line processes that are not discussed here for brevity's sake. These include several forms of coatings (normally nanometre thick metallic oxide) that improve the light transmittance and the thermal performance of the glass. Furthermore glass panels for building applications are often assembled into insulating glazing units.

14.9.5 Product permutations

Multiple treatments and processes may be applied to the same glass panel, for example, basic float glass may be clear, tinted or coated, which in turn can then be heat treated and/or bent. It can subsequently be printed, laminated and double glazed. This gives rise to a very large number of product permutations which have been increasing as new processes become available. There are, however, some permutations that are not possible, namely:

- Deeply patterned or deeply worked glass cannot be heat treated.
- Fully toughened glass cannot be subsequently surface worked or cut.

14.9.6 Glass sizes

Glass panel sizes are governed by the size of the equipment used in their production. This tends to change regularly as manufacturers and glass processors invest in larger plant.

Float glass forms the basis for all other glass products discussed in this manual. It is produced in thicknesses of 3, 4, 5, 6, 8, 10, 12, 15, 19 and 25 mm. These thicknesses can be processed into other glass products as shown in **Table 14.5**, which provides a summary of indicative panel sizes, but manufacturers and processors should be consulted for up-to-date information.

14.9.7 Tolerances and defects

Float glass can normally be cut within ±2 mm and ±4 mm of the specified length or squareness. The surface of a glass panel is not perfectly flat and it normally contains some imperfections that are measured optically in annealed glass as specified in BS EN572 (2004). In toughened glass the imperfections are limited to roller wave distortion ≤ 1 mm and overall bow ≤ 5 mm; the two can occur simultaneously and are additive.

Spontaneous fracture has been a major concern with tempered glass in the past. This is caused by nickel sulphide inclusions in the glass, which tends to expand with time thus leading to sudden fracture. This problem can be minimised by using high quality material and by heat soaking the tempered glass at 290°C for several hours as described in BS EN14179 (2005).

14.10 Conclusions

An understanding of the characteristics of structural materials, including strength, stiffness, flexibility, durability and fire resistance – as well as the potential problems and financial considerations associated with their use – is essential for good structural design and the realisation of robust structures that are fit for purpose. This chapter illustrates that, by virtue of their varying properties, frequently used structural materials are appropriate for different uses and situations, and have distinct impacts and applications for structural design.

Glass product	Maximum panel size (mm)	Comments
Monolithic annealed float glass	6000 × 3210	Glass width (3210 mm) governed by width of float bath. Lengths 6000 mm available by special order
Monolithic toughened glass	4500 × 2150 or 7000 × 1670 or 6000 × 2700	Size governed by toughening furnace which varies from one manufacturer to another. Length-to-width aspect ratio is generally limited to 1:10.
PVB laminated glass	3800 × 2400 or 4000 × 2000 or 7000 × 1800	Size limited by size of autoclave which varies from one manufacturer to another. Super size laminated glass measuring 2800 x 13000 mm is available from some manufacturers, but can be limited by size of monolithic glass used to built laminated unit.
Insulated glazing units	6000 × 2700	Limited by size of monolithic glass used to build up IGU

Table 14.5 Glass panel sizes

14.11 Acknowledgements

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CONTENTS

Chapter 15

Stability

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This chapter discusses how stability is provided for various types of building. The different actions (forces) on buildings that give rise to instability are reviewed. The effects of building 'sway' are considered. First- and second-order structural analysis is described. Low-rise steel, masonry and timber framed buildings are reviewed together with high-rise buildings constructed of steel, *in situ* and precast concrete. For each form of construction, different types of structural arrangement are discussed and typical details are shown of how stability is achieved. Various types of bracing, floor diaphragms and shear walls are discussed. Stressed skin design is also considered. Some special stability requirements relating to industrial structures, construction below ground, stability of buildings during erection, safe working practices and temporary structures are considered. Stability of aluminium structures is reviewed.

15.1 Introduction

The stability of a building is its ability to satisfactorily resist horizontal or other disturbing actions or forces which could cause overturning, uplift or cause the building to sway. Stability is one of the most important aspects of the structural design of buildings. In order to design for stability, the engineer must have a clear understanding of how the horizontal or disturbing actions (forces) are transferred to the building foundations. The load paths should be clearly defined and should be as direct as possible. Typically the roof or floors of a building will act as diaphragms transmitting the horizontal actions, for example, wind on cladding, to vertical braced frames or shear walls and then to the foundations. Design should ensure that adequate horizontal and vertical framing exists to prevent sway and transmit these actions.

15.2 General considerations

The following are general stability considerations which apply to different types of building.

15.2.1 Actions to be considered

Below are examples of loads that may impose lateral actions on the structure:

- Wind loads
- Crane and machinery loads
- Earthquake
- Geometrical imperfections in the framing (sway stability)
- Horizontal component of soil, water loads and drifted snow
- Accidental loads (vehicle impact, explosions).

When designing to ensure stability, partial load factors (BS EN1990: 2002) must be applied to the lateral actions (overturning) and to the restoring actions. To ensure an adequate factor of safety in the design low partial load factors are applied to the restoring actions (**Figure 15.1**).

For a more detailed discussion on loading, see Chapter 10: *Loading*.

$F_{bldg,k}$ – Characteristic dead wt. of building (including foundations)

$F_{w,k}$ – Characteristic wind force (ignoring the effects of sway)

γ_G – Partial factor for permanent loads (0.9)

γ_Q – Partial factor for variable loads (1.5)

Taking moments about 'X'

Design overturning moment (OM) = $F_{w,k}\gamma_Q H/2 = F_{w,k}1.5H/2$

Design restoring moment (RM) = $F_{bldg,k}\gamma_G L/2 = F_{bldg,k}0.9L/2$

OM < RM

15.2.2 Structural arrangement

The structure should be designed to resist the horizontal actions in two orthogonal directions by means of points of restraint or braced bays. Horizontal wind loads or other lateral actions are transferred to the foundations by various means as follows:

- The diaphragm action of floors or walls acting as plates or shear walls.
- Horizontal and vertical bracing carrying the lateral actions as axial load in triangulated frames.
- Moment frames with 'pinned' connections at the supports.
- Vertical cantilever columns with 'fixed' base connections at foundation level.
- Vertical cantilever concrete core walls (enclosing lifts, stairs or service ducts).
- Buttressing by means of diaphragm or fin walls.

Where movement joints are provided in a structure each part of the structure must be independently stable with its own provision for stability.

Points of restraint or shear walls should be arranged on plan so that the centre of shear coincides with the resultant horizontal action. Where this is not possible the resulting eccentric

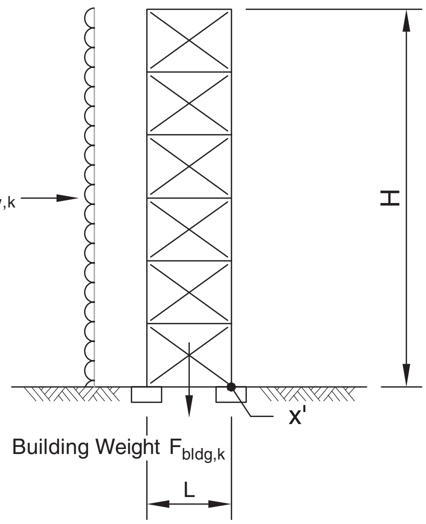


Figure 15.1 Ultimate limit state (ULS) of equilibrium – overturning

moment should be distributed between shear walls (refer to example in 15.3.2.3).

15.2.3 Sway effects

When a structure is loaded vertically it should have sufficient stiffness not to cause excessive lateral deformations or sway. The sway can be caused by geometrical imperfections in the framing, i.e. out of plumb. The Eurocodes (BS EN1992:2004, BS EN1993:2005, BS EN1995:2004, BS EN1996:2005) recommend an *equivalent horizontal force* (EHF) is determined and added to the other horizontal actions applied to the building. For example, total horizontal action applied to the building is the sum of the wind plus EHF (**Figure 15.2**). Refer to section 15.4.5 below.

15.2.4 First- and second-order structural analysis

A first-order structural analysis is an analysis where the frame deformation is calculated on a single pass basis and bending moments and shear forces are determined on that basis. In a second-order analysis account is taken of the frame deformation in the analysis and the frame deflections, bending moments and shear forces are amended accordingly. This is the P-δ effect.

If the frame is sufficiently stiff then the second-order effects can usually be ignored. But in some frames such as multi-storey moment frames, the additional moments can be significant and should be taken into account. EC3 Cl 5.2.1(2) gives recommendations when second-order analysis should be adopted. If the frame is sensitive to second-order effects then all the lateral actions (e.g. wind + EHF) are increased or amplified.

Many modern-day computer analysis programs carry out first- and second-order analysis including automatic allowance for frame imperfections. Consequently, the designer is not required to calculate and apply EHFs. However, it is

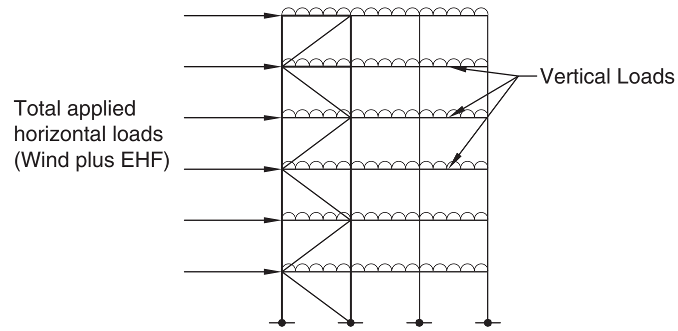


Figure 15.2 Horizontal loads applied to bracing

recommended that the designer carries out sufficient hand calculations to validate the computer output.

15.2.5 Building Regulations

All building construction in England and Wales has to comply with the Building Regulations (HM Government, 2000). (Similar regulations exist for other parts of the UK.) Part A Structure of the Regulations stipulates the requirements for stability, namely:

Loading

> A1 (1) The building shall be constructed so that the combined dead, imposed and wind loads are sustained and transmitted by it to the ground:
>
> (a) safely; and
>
> (b) without causing such deflection or deformation of any part of the building, or such movement of the ground, as will impair the stability of any part of another building.
>
> (2) In assessing whether a building complies with sub paragraph (1) regard shall be had to the imposed and wind loads to which it is likely to be subjected in the ordinary course of its use for the purpose for which it is intended.

15.3 Low-rise buildings

Low-rise buildings are generally accepted to be one, two or three storeys in height. In the UK, most single storey buildings are either of steel or masonry construction.

15.3.1 Steel framed single storey buildings

Lateral stability of single storey steel framed buildings should be provided in two orthogonal directions by the following means:

- rigid frames (e.g. portal frames) and/or
- braced bays (plan bracing in the roof acting in conjunction with vertical bracing in the walls).

15.3.1.1 Portal action and bracing

Wind is the principal horizontal action requiring to be resisted by a single storey building. Wind pressure and suction on opposite sides of a building cause a resultant transverse action on the building. Similarly, wind pressure differences between the gables together with longitudinal frictional drag on the building due to wind cause longitudinal actions on the building. Single storey industrial buildings which contain an overhead travelling crane also have to resist transverse and longitudinal actions caused by crane surge.

The transverse actions acting on a single storey steel framed building are usually resisted by frame action, for example, portal frames or vertical cantilever columns (**Figure 15.3**). The frames or cantilever columns are typically at say 6–8 m centres. In the longitudinal direction the resultant horizontal actions are resisted by horizontal and vertical braced bays. Wind pressures and suctions on the gables are resisted by columns spanning vertically. These columns are supported at eaves level by a gable wind girder which transmits the wind actions to the vertical bracing which in turn conveys the actions to the foundations (**Figure 15.4**). In the case of low-pitch portal frame buildings, the gable columns usually span to a wind girder at rafter level (**Figure 15.5**).

Where a lattice girder or truss exists as the roof structure (**Figure 15.3**), the eaves wind girders at each end of the building can be connected by means of tie members to prevent instability of the lattice girders (**Figure 15.6**). This can occur under wind uplift conditions causing compression in the bottom chord of the lattice girders.

In single storey steel frame construction braced, bays are typically provided in the end bays to facilitate erection. Also for material efficiency the braced bay should be adjacent to the

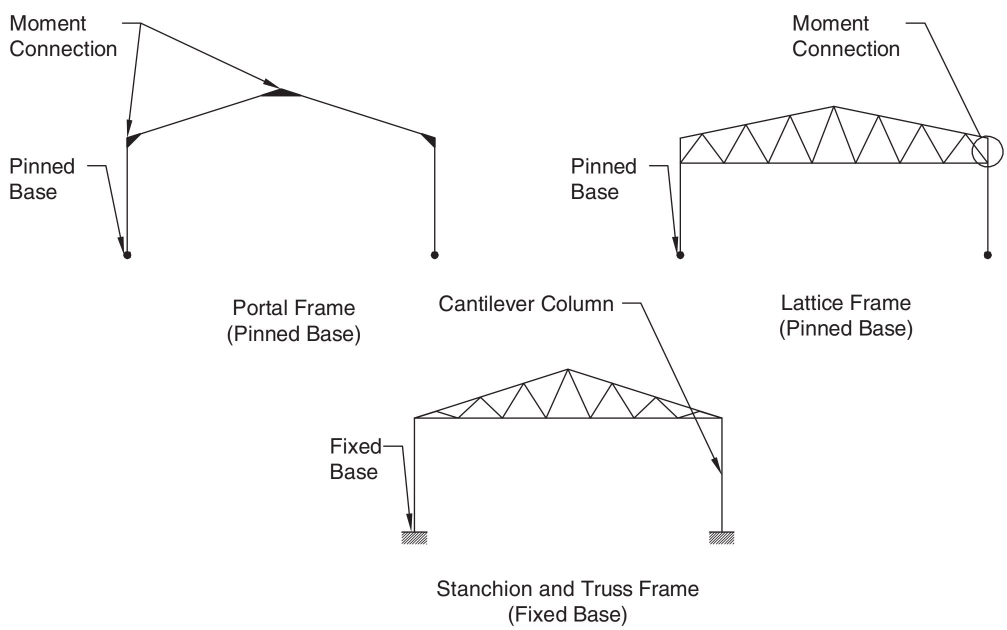


Figure 15.3 Types of steel frame

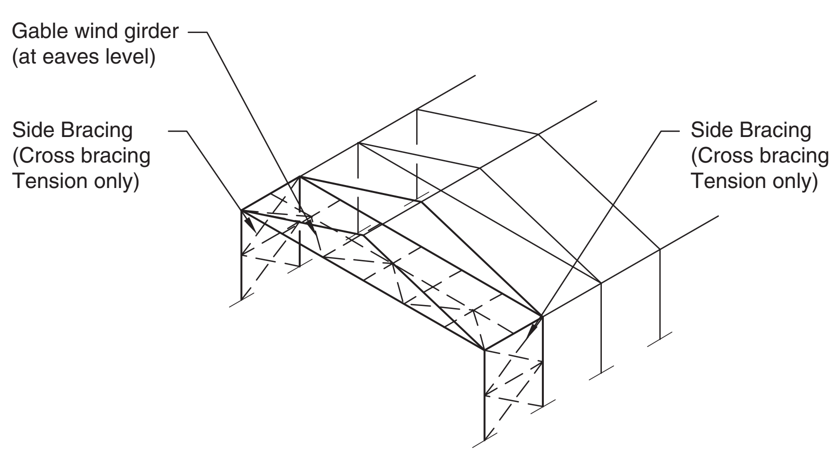


Figure 15.4 Bracing for single storey steel frame building

gable wind girder. Generally braced bays should be provided at least every eight bays (**Figure 15.7**).

Common forms of bracing for single storey steel frames are shown in **Figure 15.8**. The bracing members are typically rolled hollow sections, angles or flats. For economy it is usual to incline the bracing members at approximately 45°. A single brace is designed to take compression and tension whereas cross bracing members are usually designed for tension only. Where a bracing member cannot be permitted in a particular bay, if, for example, the bracing member would encroach on a door opening, the bracing member can either be transferred to another bay or it can be replaced with a portal frame brace (**Figure 15.8**). If a portal framed brace is used then the serviceability limitation should be checked since this form of bracing is significantly less stiff than a braced bay.

15.3.1.2 Stressed-skin design

Single storey steel framed buildings are often clad with profiled metal sheeting. Examples of such buildings are factories, warehouses, retail premises and sports centres. With such buildings the cladding can be utilised to brace the steel frame and assist in providing lateral stability. The cladding can also be used to provide restraint to the compression flange of the steel frame members. In this way economies can be made in design of the steel frame.

Wall and roof cladding will stiffen and strengthen the structure and will reduce the deflections of the bare steel frame. Stressed-skin design is the evaluation of the shear diaphragm resistance that the roof and side sheeting panels can provide or contribute to the lateral stability of the building. The sheeting panels must be securely fixed to the secondary members (purlins) by mechanical fasteners through the trough of the sheeting. In the case of low pitch portal frame buildings, wind loading from the gable posts is transmitted to the side braced bays by the diaphragm action of the roof sheeting (**Figure 15.9**).

Careful specification and detailing of the fixings for the roof sheeting is required in order to ensure proper diaphragm action occurs. The following must also be considered in the design:

- Side walls must be adequately braced to transmit horizontal loads to the foundations.
- Sheeting panels must be adequately fixed to the secondary members (purlins).

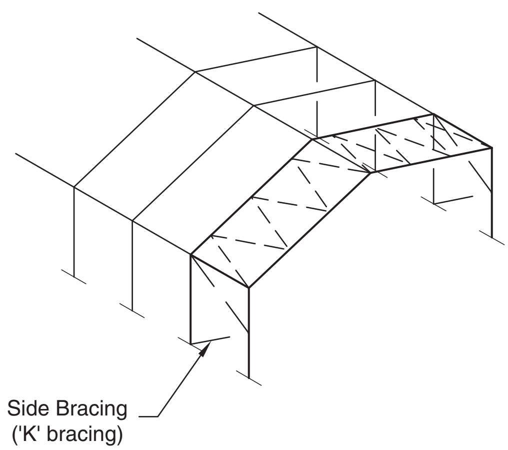


Figure 15.5 Portal frame bracing

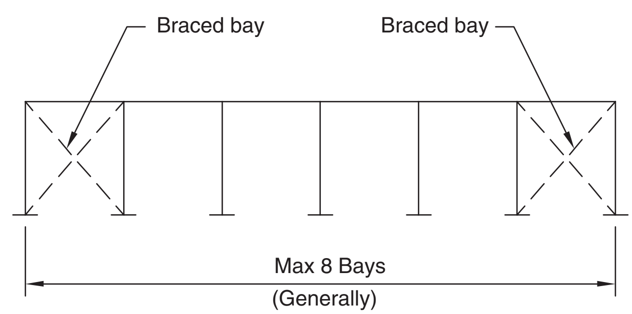


Figure 15.7 Side elevation arrangement of braced bays

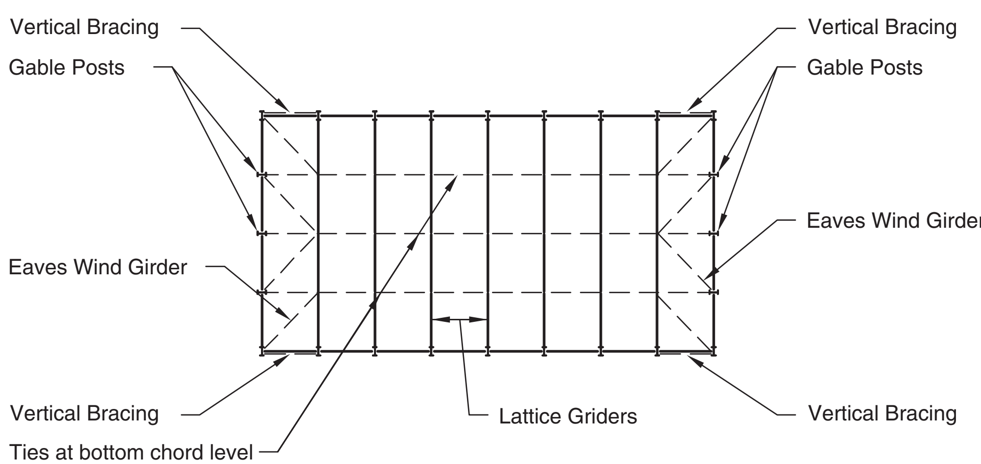


Figure 15.6 Plan bracing layout at bottom chord level of lattice girder

- Sufficient fixings must be installed where laps occur in the sheeting to ensure continuity.
- Flange forces in edge members resulting from the diaphragm action must be considered.
- The diaphragm actions must be transmitted to the main frame via suitable structural connections.

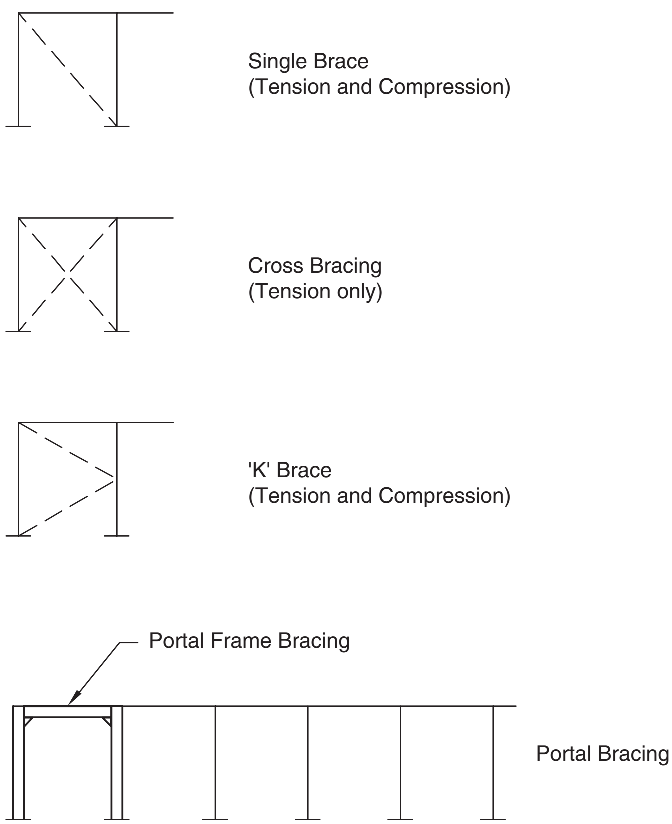


Figure 15.8 Types of bracing for single storey steel buildings

- The structure must be adequately braced during erection.
- Sheeting panels must not be removed without full consideration of the effects.

For detail recommendations for stressed skin design refer to BS EN1993-1-3:2006.

15.3.2 Low-rise masonry structures

15.3.2.1 Lateral stability of masonry structures

Lateral stability of low-rise masonry structures is usually achieved by means of cross wall construction. Lateral wind load on the side or gable of a building is transmitted to the floor or roof structure. The floor or roof acts as a stiff plate or diaphragm and the wind loads are in turn transferred to the gable or side walls and then to the foundations (**Figure 15.10**). If the roof or floor is an *in situ* reinforced concrete slab cast on the inner leaf of a cavity wall then the slab will act as a diaphragm and usually no further checks are required. If the roof or floor consists of pre-stressed concrete planks then lateral and longitudinal ties will need to be incorporated in an *in situ* structural topping in the form of a light steel fabric. Where the planks run parallel to the cross walls anchor ties will also be needed. A lightweight steel or timber roof will require bracing members and anchors to transmit the lateral wind loads from the diaphragm to the gable or cross walls (**Figure 15.11**).

For more on stability in masonry design, please see Chapter 20: *Masonry*.

Cross wall construction

The cross walls (gable walls in **Figure 15.10**) act as shear walls and should preferably be positioned parallel to the direction of loading. The layout of the walls should provide lateral stability for the building in two orthogonal directions. Referring to **Figure 15.12**, plan arrangement A is unstable in the longitudinal direction since there are no longitudinal walls

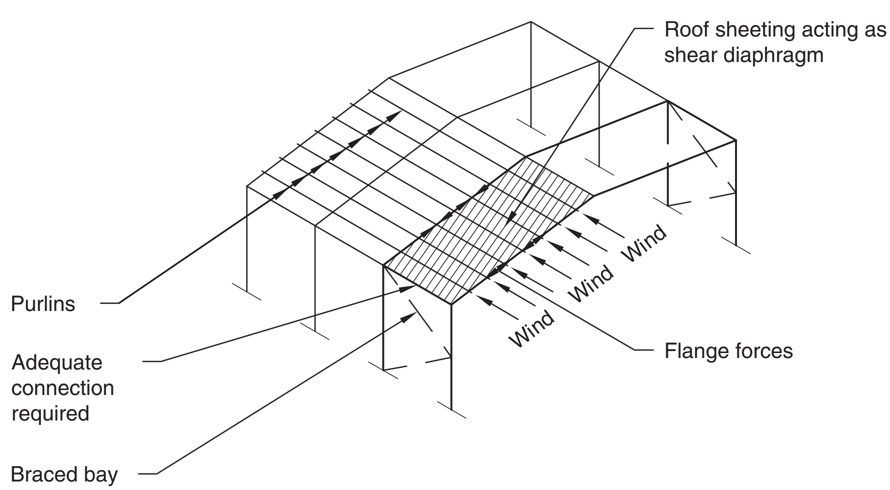


Figure 15.9 Stressed-skin gable bracing

or spine walls resisting the wind load. Plan arrangement B on the other hand, has shear walls in two directions and is therefore stable.

A common form of cross wall construction is shown in **Figure 15.13**. This layout has a long rectangular floor plan with a repetitive arrangement of dividing walls. Many school buildings, hotel bedroom blocks and office buildings have this basic arrangement. The cross walls provide lateral stability when considering wind in the transverse direction and the corridor or spine walls provide stability in the longitudinal direction when wind is acting on the gables.

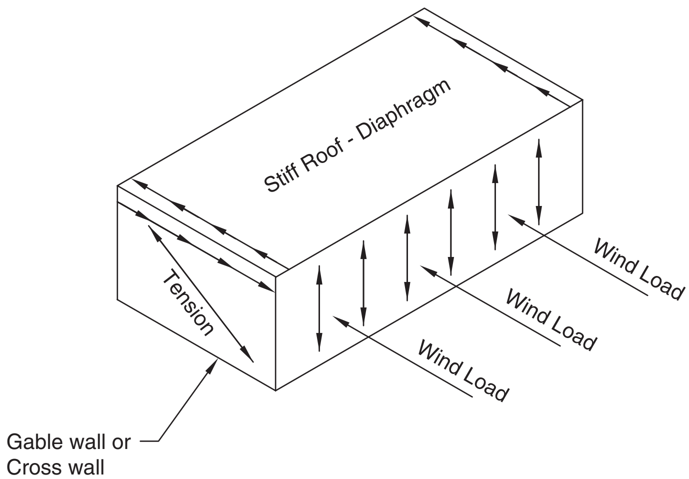


Figure 15.10 Wind acting on side of building

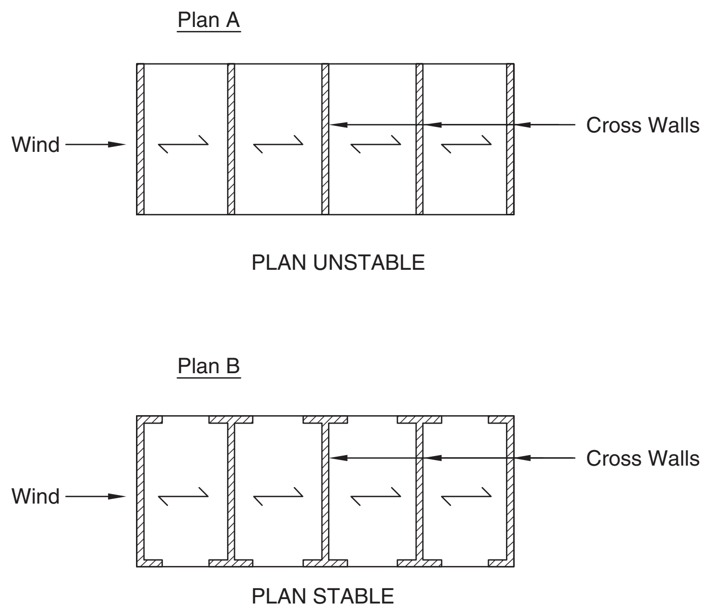


Figure 15.12 Unstable and stable layout of walls

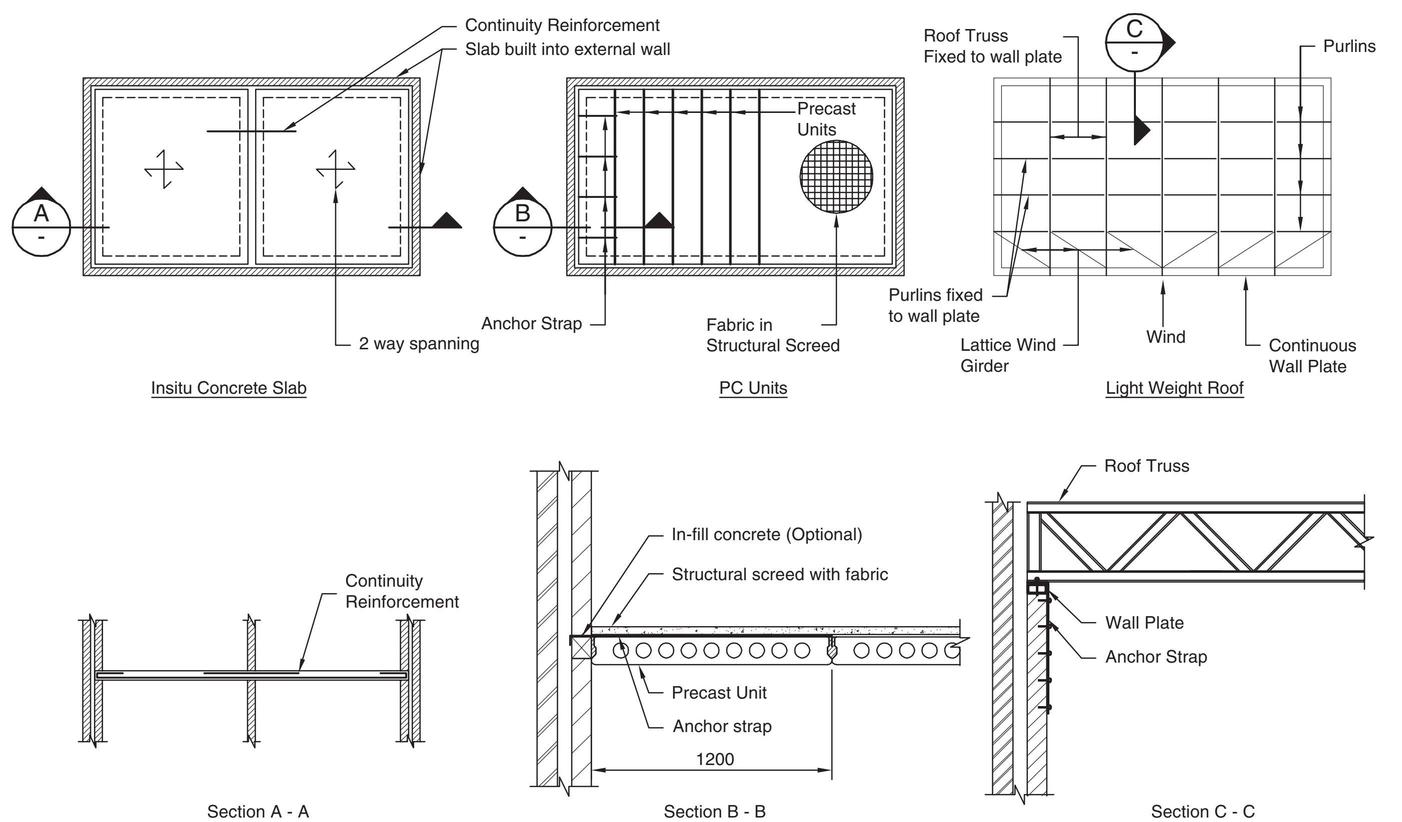


Figure 15.11 Forms of roof or floor construction acting as a diaphragm

Cellular block construction

A further arrangement of cross walls in low-rise construction is the cellular block plan shown below in **Figure 15.14**. Load-bearing walls arranged orthogonally on plan provide a robust and extremely stable form of construction.

15.3.2.2 Wind load carried by symmetrical arrangement of shear walls

The distribution of wind load carried by a symmetrical arrangement of shear walls is readily determined and is proportional to the stiffness of the walls. **Figure 15.15** and the example below demonstrate the principle involved.

Design wind load on building $= F_d$

Total moment of inertia of resisting walls $\Sigma I = I_A + I_B + I_C$

Therefore design shear carried by wall (A) $F_{Ad} = \dfrac{F_d I_A}{\Sigma I}$

Where $I_A = t\ell_A^3/12$

Design shear carried by wall (B) $F_{Bd} = \dfrac{F_d I_B}{\Sigma I}$

Where $I_B = t\ell_B^3/12$

Design shear carried by wall (C) F_{Cd} is the same as wall (A)

15.3.2.3 Wind load carried by an asymmetrical arrangement of shear walls

When shear walls are arranged in an asymmetrical layout the line of action of the applied wind load is no longer coincident with the shear centre. As a consequence, a twisting action is applied to the shear walls. The distribution of wind load to the shear walls is therefore not only dependent upon the relative stiffness of the walls but also the twisting moment from the applied wind load caused by the eccentric shear centre. The principles involved are shown in **Figure 15.16**.

The design wind load F_d acting on the centre line of the building can be replaced by F_d acting through the shear centre together with a twisting moment $F_d e$ as shown in **Figure 15.16**. Since the load F_d is now applied through the shear centre, the wind action is distributed between the shear walls in proportion to the wall stiffness as follows:

Design shear carried by wall (A) $F_{Ad} = \dfrac{F_d I_A}{(I_A + I_B + I_C)}$

$= \dfrac{F_d I_A}{\Sigma I}$

Design shear carried by wall (B) $F_{Bd} = \dfrac{F_d I_B}{\Sigma I}$

and by wall (C) $F_{Cd} = \dfrac{F_d I_C}{\Sigma I}$

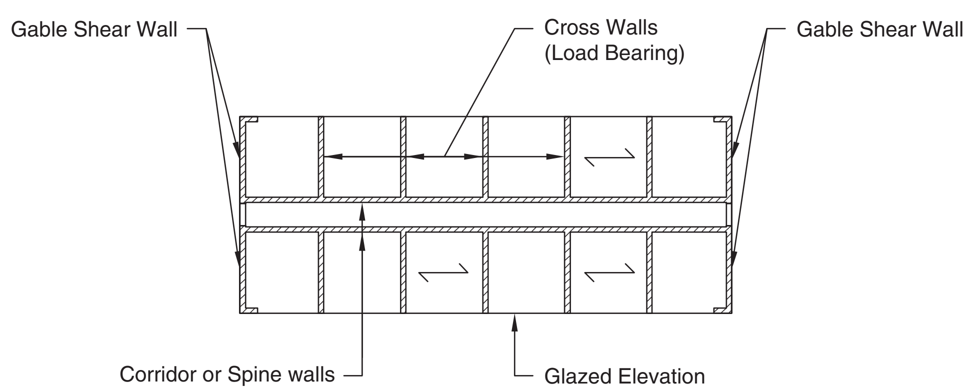


Figure 15.13 Layout plan showing cross wall construction

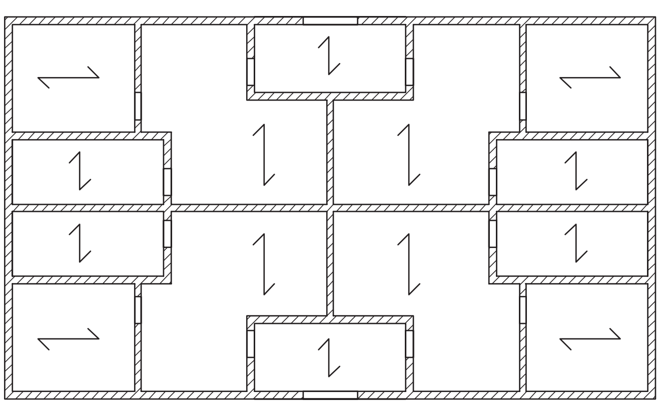


Figure 15.14 Cellular block plan

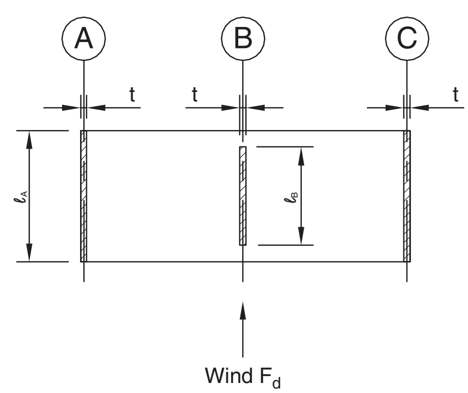


Figure 15.15 Symmetrical arrangement of shear walls resisting wind load

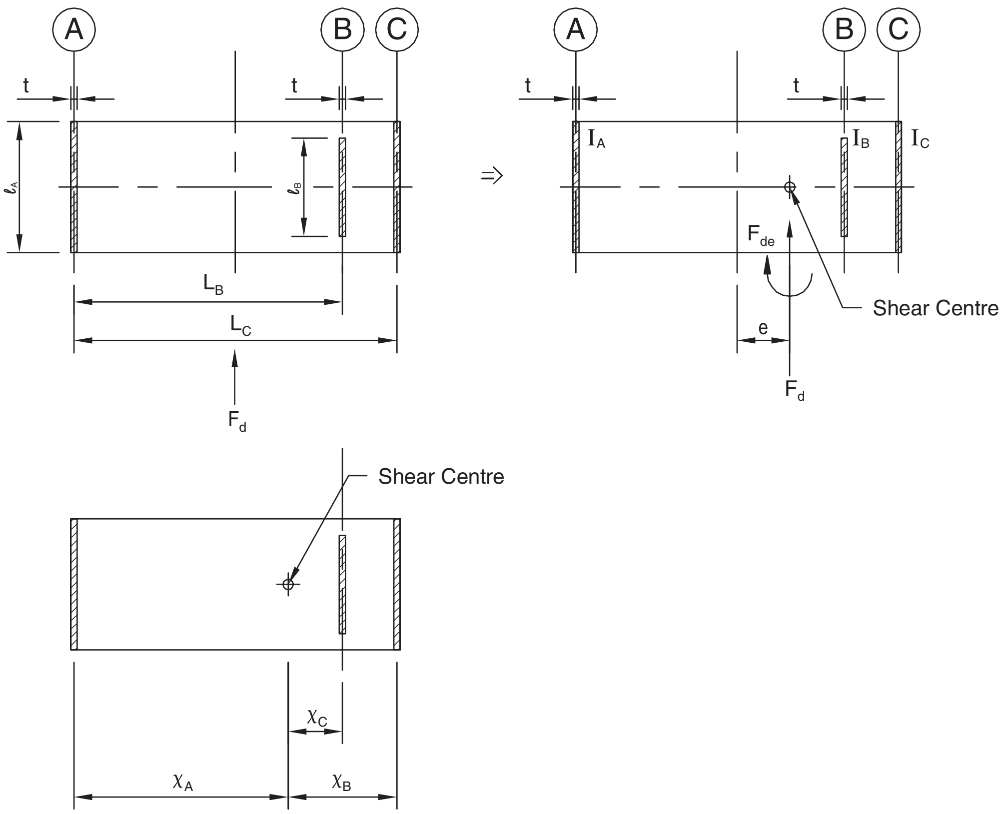


Figure 15.16 Asymmetrical arrangement of shear walls resisting wind load

The twisting moment ($F_d e$) causes an additional load on the walls of F_{Ad}, F_{Bd} and F_{Cd}. In this case, the shear load in wall A is increased and the shear in walls B and C is reduced due to the twisting moment. The floor is assumed to be rigid.

Total moment of inertia of resisting walls $\Sigma I = I_A + I_B + I_C$

$$\text{Pos'n of shear centre } x_a = \frac{I_B L_B + I_C L_C}{\Sigma I}$$

Additional load applied to

$$\text{wall (A) } F_{Ad}{}^* = \frac{F_d e x_a I_A}{(I_A x_a^2 + I_B x_b^2 + I_C x_c^2)}$$

$$= \frac{F_d e x_a I_A}{\Sigma I x^2}$$

The total shear carried by wall (A) is therefore the algebraic sum of loads due to direct shear and twisting

$$F_{Ad} = \frac{F_d I_A}{\Sigma I} + \frac{F_d e x_a I_A}{\Sigma I}$$

The shear load resisted by each wall can be expressed as,

$$F_{nd} = \frac{F_d I_n}{\Sigma I} \pm \frac{F_d e x_n I_n}{\Sigma I x^2}$$

* For detailed explanation of load distribution between asymmetrical shear walls refer to Hendry *et al*. (2004).

15.3.2.4 Design of masonry shear walls

Wind acting on a building elevation is transmitted via stiff floor diaphragms to the cross walls or shear walls. The shear walls provide stability to the building by acting as vertical cantilevers (**Figure 15.17**).

Vertical load resistance

At the ultimate limit state (ULS), the design value of the applied vertical load N_{Ed} should be less than or equal to the design value of the vertical resistance of the wall N_{Rd}.

$$N_{Ed} \le N_{Rd}$$

$$N_{Rd} = \phi t f_d \text{ (per unit length of wall)}$$

Where $$N_{Ed} = \frac{Nid}{l} + \frac{Mid}{l^2/6}$$

And N_{id} – design value of vertical load per unit length of wall

M_{id} – design value of bending moment due to lateral load

ℓ – length of wall

t – thickness of wall

ϕ – capacity reduction factor (ϕ can be determined at the bottom or the middle of the wall as appropriate)

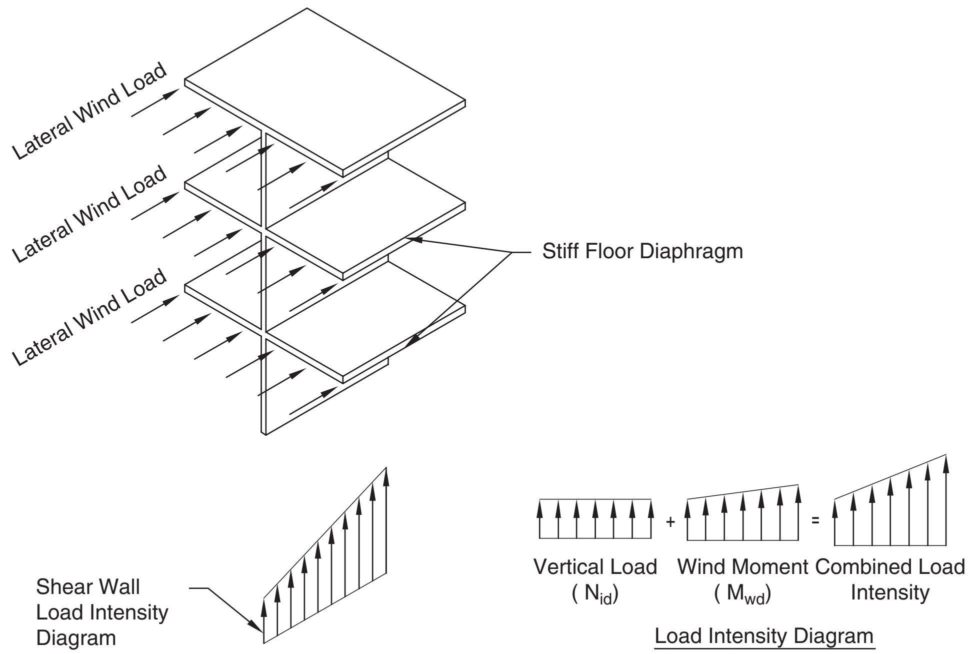


Figure 15.17 Shear wall acting as vertical cantilever

Three checks are normally carried out:

(i) Stability check – maximum wind, minimum DL ($\gamma_G = 0.9$, $\gamma_Q = 1.5$).

(ii) Wall resistance at base of ground floor wall ($\gamma_G = 1.35$, $\gamma_Q = 1.5$, $\gamma_Q = \psi 1.5$).

(iii) Wall resistance at mid-height of ground floor wall ($\gamma_G = 1.35$, $\gamma_Q = 1.5$, $\gamma_Q = \psi 1.5$) where wind overturning moment and vertical load are reduced but wall resistance is also reduced at wall mid-height by capacity reduction factor ϕ.

Where an intersecting wall is used as the shear wall flange the connection between the two walls should be checked for vertical shear. Where metal ties or similar connectors are used to bond two walls together the appropriate characteristic shear strength of the connectors should be used.

Horizontal shear resistance

At the ULS, the design value of the applied shear load V_{Ed} should be less than or equal to the design value of the shear V_{Rd},

where $V_{Rd} = f_{vd}\, t\, \ell_c$

and f_{vd} – design value of shear strength of masonry

t – thickness of wall resisting shear

ℓ_c – length of compressed part of wall, ignoring any part of the wall that is in tension

15.3.3 Low-rise timber structures

15.3.3.1 Stability of timber structures

Stability of low-rise timber structures is provided in a similar way to other forms of construction, notably the lateral actions such as wind forces on the structure are resisted by braced panels in two orthogonal directions. In addition, overall stability checks should be carried out for the following:

- Overturning
- Sliding
- Uplift.

Since timber structures are light in weight these checks are particularly important and can be critical when the building height to breadth ratio exceeds 2:1. Stability checks need to be carried out not only for the final built condition but also for the construction phase when for example roof trusses and roof tiles are not in place.

Figure 15.18 shows lateral forces applied to the gable elevation of a two-storey timber framed building. The lateral forces from the masonry or timber cladding are transferred to horizontal timber bracing at roof truss tie level and to the stiff diaphragm at first floor level. These horizontal loads are then transmitted to the stiff wall diaphragms which in turn carry the loads to the foundations. The wall diaphragms also support the vertical loads from the roof and first floor in addition to providing the in-plane shear or racking resistance to the lateral actions. The wall panels also resist wind loading perpendicular to the panels.

15.3.3.2 Stiff horizontal diaphragms

Flat roofs and timber panel floors are used to transfer lateral actions to stiff vertical wall panels. Eurocode 5 provides some simple guidelines based on experience and practice to demonstrate that a conventional floor or flat roof consisting of wood-based panels fixed with nails or screws to timber joists can be assumed to have adequate strength and stiffness to act as a horizontal diaphragm. The approach adopted assumes the timber deck to act as a deep beam. The long edges (supported on wall plates or similar) are assumed to be the beam flanges which resist the horizontal bending moment. The decking is the beam web which transmits the shear force to the supporting shear walls (**Figure 15.19**).

The guidelines are as follows:

- The span ℓ must lie between 2b and 6b.
- The critical ultimate design condition must be failure of the fasteners and not failure of the panels.
- Edge beams should be designed to resist the maximum bending moment in the diaphragm.
- The panels are fixed to the supporting edge beams and joists. All unsupported edges should be connected to adjacent panels by fixing to battens in accordance with Eurocode 5, 10.8.1.
- The fixings should be nails (other than smooth nails) or screws at a maximum spacing of 150 mm along panel edges and 300 mm spacing along the supports.

The following checks are required to be carried out:

a) Bending strength of 'flanges'

$$\frac{F_d l}{8(wh)b} \leq f_{t,0,d}$$

Where F_d	– total design force on diaphragm (N)
Span of panel	– ℓ (mm)
Width of panel	– b (mm)
edge beam width	– w (mm)
ditto height	– h (mm)
$f_{t,0,d}$	– design tension strength of timber edge beam
	– $k_{mod}\ f_{t,0,k}/\gamma_m$
k_{mod}	– modification factor for load duration (ref. Table 3.1, Eurocode 5)
$f_{t,0,k}$	– characteristic tensile strength of timber
γ_m	– partial coefficient for material properties (ref. Table 2.3, Eurocode 5)

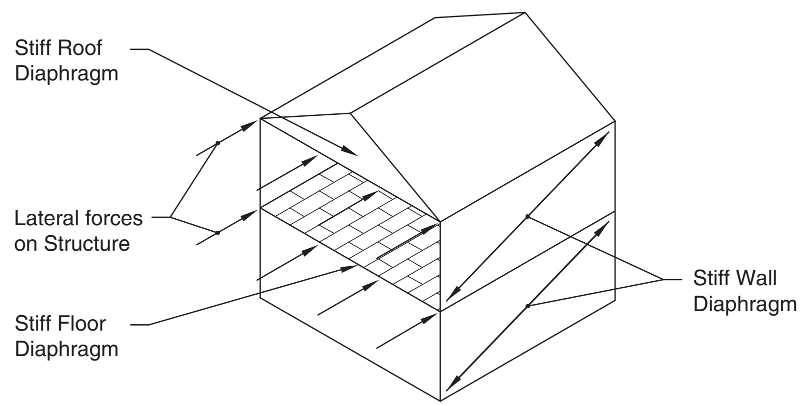


Figure 15.18 Lateral forces carried by two storey timber framed structure

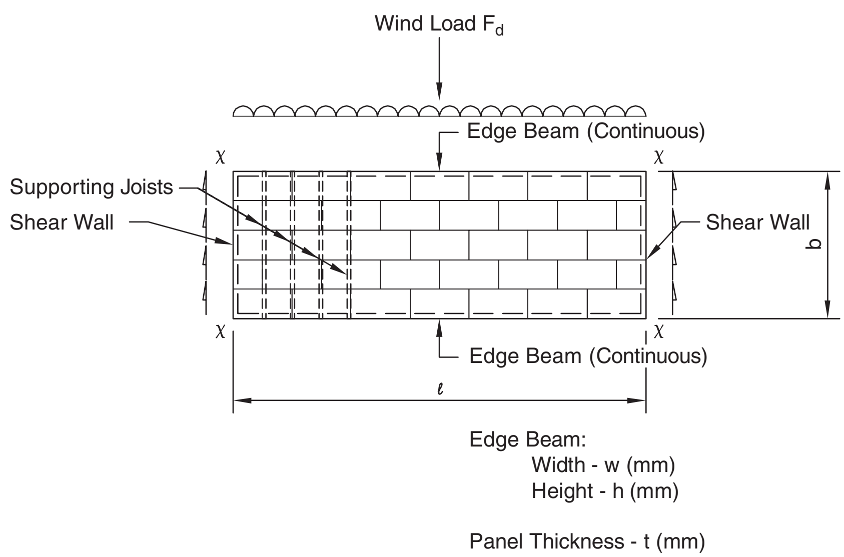


Figure 15.19 Plan of horizontal diaphragm

The edge beam must be continuous.

b) Shear strength of 'web'

$$\frac{F_d}{2bt} \le f_{v,d}$$

Where t – thickness of panel (mm)

$f_{v,d}$ – design panel shear strength of decking (N/mm²)

$$= \frac{k_{mod}\, f_{v,k}}{\gamma_m}$$

$f_{v,k}$ – characteristic panel shear strength

c) Shear strength of panel support fixings (along XX)

$$F_d \le \frac{2b\, R_d}{s}$$

Where R_d – design shear resistance of fastener (N)

s – spacing of fasteners (mm)

15.3.3.3 Stiff vertical diaphragms

The stiff vertical wall diaphragms used in timber framed structures are constructed from wood-based sheet material mechanically fixed (using nails and screws) to a timber frame. The racking resistance provided by the panel is developed primarily by the perimeter panel fixings (**Figure 15.20**). The panel is fixed to the sole plate by bolting or other suitable anchorages to prevent sliding. The design must also ensure that an adequate factor of safety exists to prevent overturning. Eurocode 5, 9.2.4.1 provides detailed design guidance.

15.3.3.4 Trussed rafter roofs and stability bracing

Trussed rafters are invariably part of a timber framed house or residential development and are usually designed and manufactured by specialist suppliers. The trussed rafters are designed to support the weight of the roof covering, ceiling loads including imposed loads and water tanks, etc. They must of course also be designed to resist wind loading. Spans of up to 25 m can be achieved. Trussed rafters are manufactured from various timber grades and the triangulated frameworks are assembled together using punched metal plate connectors.

For trussed rafters to function properly as designed and to ensure overall stability of the complete roof structure they require the addition of stability bracing. These bracing members serve two functions: they prevent lateral instability of the compression members within the trussed rafter by increasing their buckling strength; and they also provide overall stiffness to the roof structure and assist in transferring the lateral loads on the roof structure to the side or gable walls. **Figure 15.21** shows a typical trussed rafter roof with stability bracing.

Whilst the truss rafter supplier is responsible for the design of the truss based on the loads provided, it is the building designer who is responsible for the stability of the roof structure overall. The building designer should make sure that the bracing is adequate to ensure the overall stability of the whole roof structure and supporting walls and that the roof is properly fixed to the vertical wall components to resist uplift forces.

15.4 Multi-storey buildings

15.4.1 Lateral stability of multi-storey steel and concrete buildings

Lateral stability of multi-storey buildings, be they steel or concrete construction, depends upon the provision of either braced bays or moment frames within the building. In the UK, by far the majority of buildings depend on braced bays for lateral stability. That is, they depend on either vertical steel bracing or reinforced concrete shear walls to provide stability and to prevent sway of the building frame. The stability requirements of multi-storey buildings are similar in principal to low-rise structures as follows:

- Floors and roof members are required to act as stiff diaphragms.
- Braced bays (vertical bracing or shear walls) are required in two directions at right angles.

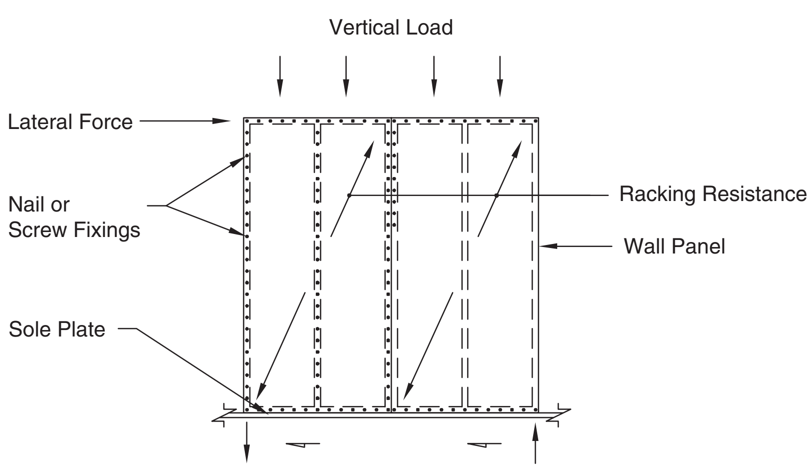
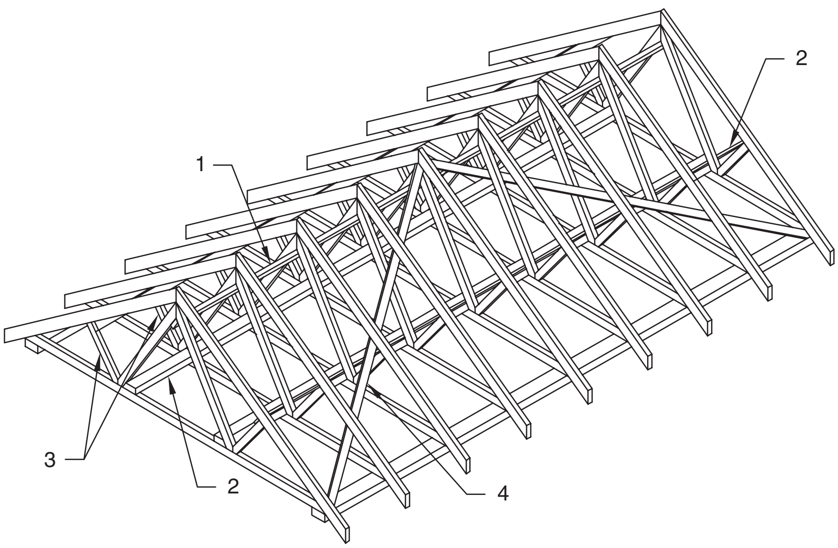


Figure 15.20 Typical wall panel resisting lateral and vertical load

- The braced bays are required to be continuous for the full height of the building.
- If a braced bay is interrupted for any reason then the forces carried by the bracing or shear wall should be transferred to other braced bays.
- If movement joints are present then each portion of the building must be stable within itself.
- The braced bays should be arranged such that the lateral force acting on the building should coincide with the centre of shear.

Where the braced bays do not coincide with the shear centre then torsion effects should be taken into account. As far as possible the braced bays should be arranged at the extremities of the building to resist the torsion effects. The loads carried by an asymmetric arrangement of bracing or shear walls can be determined by hand calculation (refer to Section 15.3.2.3). Alternatively the loads can be determined by modelling the bracing system as a stiff beam supported by spring supports with a stiffness k representative of the vertical bracing or shear wall stiffness (units of k are typically kN/mm) (**Figure 15.22**).



1. Longitudinal runner at apex
2. Longitudinal runners at intermediate nodes, may be omitted if this does not leave more than 4.2 m of unbraced rafter or more thar 3.7 m of unbraced ceiling tie.
3. Further bracing is required on these internal members for spans over 8 m in duopitch roofs, 5 m in monopitch roofs.
4. Under-rafter diagonal brace at approximately 45° to the rafters.

Figure 15.21 Trussed rafter stability bracing. Reproduced from TRADA Technology (2006) with permission

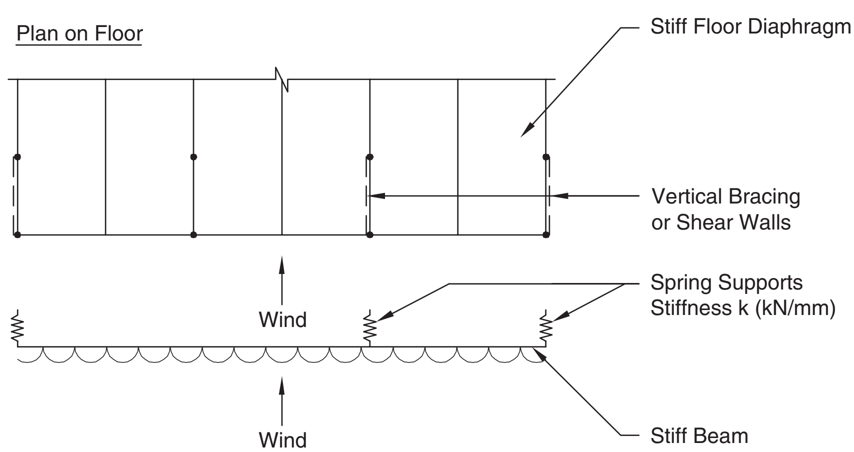


Figure 15.22 Distribution of wind load on asymmetric layout of bracing

15.4.2 Design of floor diaphragms

The design of a floor diaphragm is based on a deep beam analogy. The top and bottom chords usually consist of *in situ* edge strips of reinforced concrete along the two main sides. The top and bottom chords together provide the bending moment of resistance. Shear in the deep beam is carried by the floor units and is transmitted to the shear walls or bracing along the sides of the floor (**Figure 15.23**).

The floor diaphragms can be designed either with or without a structural topping. If no structural topping is provided then a suitable precast concrete (PC) floor unit is the type shown in **Figure 15.24**. The joint between the units is filled with low-shrink grout. Shear is transmitted across the joint by compression of the grout and aggregate interlock. It is most important that the joint is properly filled with appropriate grout. The shear stress in the joint is a function of the effective joint depth. The joint depth h_j is normally taken as (h – 35) where h is the overall depth of the unit (refer to **Figure 15.24** and Eurocode 2, 10.9.3). It is also imperative that the units are held tightly together and so in the case of a concrete structure a further *in situ* strip is provided along the sides of the floor to retain the units. The sides of the floor are the regions of highest shear where the shear stresses in the floor are transferred to the shear wall (**Figure 15.23**). Continuity reinforcement is required here between the *in situ* strip and the shear wall. In the case of a steel framed building the PC units can be retained by an angle frame bolted or welded to the steelwork.

If any of the above requirements cannot be achieved then a structural topping should be provided. This should take the form of a fine aggregate concrete reinforced with a structural fabric. The minimum thickness of topping is usually around 50 mm and is based on the topping thickness at mid-span (to take account of camber in the units).

For more on designing structural elements in concrete, see Chapter 17.

15.4.3 Steelwork construction

15.4.3.1 Vertical bracing

With steelwork construction the vertical braced bays usually take one of the following forms:

- Vertical triangulated steel bracing, comprising: hollow sections, angles, flats or channels. The bracing members are either single diagonals (typically hollow section tubes) acting in tension or compression, or crossed members (typically angles or flats) acting as tension only members. The bracing members are usually inclined at approximately 45° for economy (**Figure 15.25**).
- Concrete shear walls – lift shafts, stair wells or service cores (**Figure 15.27**).

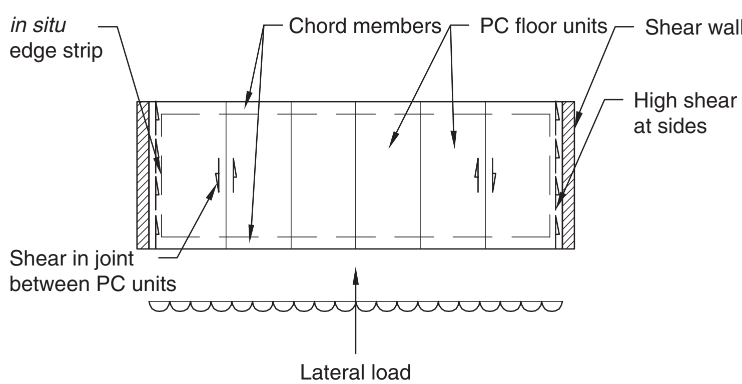


Figure 15.23 PC concrete units acting as floor diaphragm

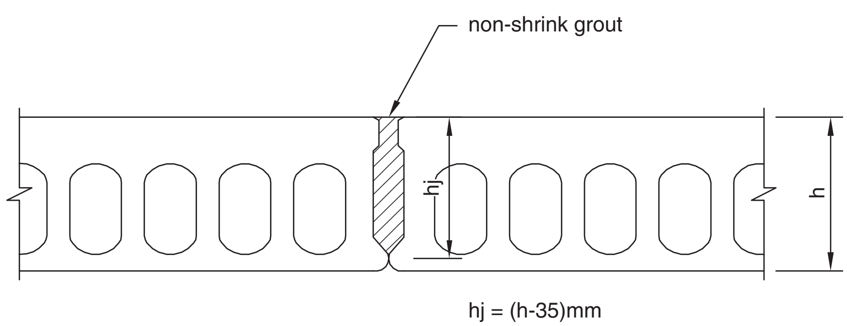


Figure 15.24 Joint between PC units

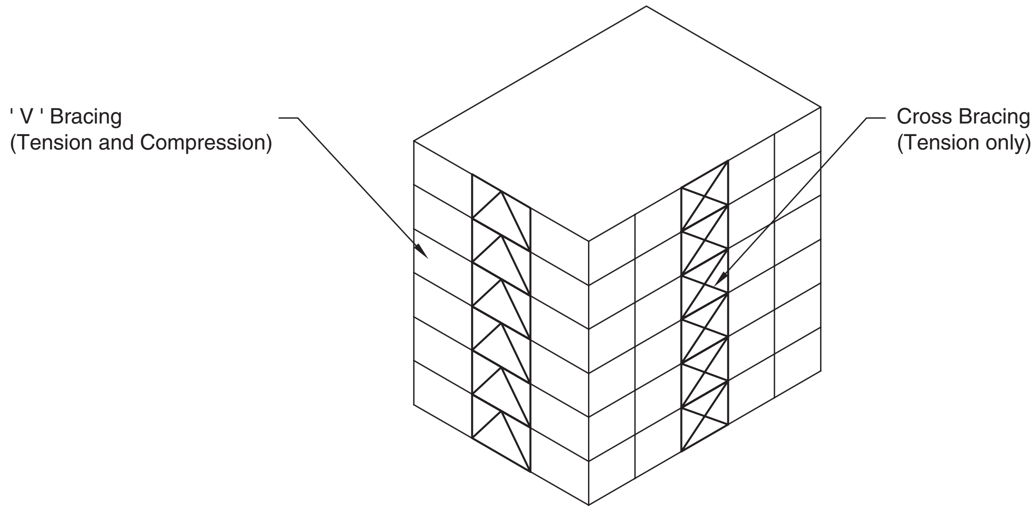


Figure 15.25 Steel framed building showing types of vertical bracing

Figure 15.26 Vertical bracing to multi-storey steel framed building. Courtesy of Elland Steel Structures Ltd

15.4.3.2 Design of vertical bracing systems

Vertical bracing should be designed for the following forces:

- Wind load or other lateral load.
- Equivalent horizontal forces (EHF) – forces that replicate the effects of vertical sway due to frame imperfections.
- If the frame is flexible, i.e. if the frame is 'sway sensitive', then the above lateral loads should be amplified to take account of second order effects (i.e. $P - \delta$).

Reference should be made to Eurocode 3 Cl 5.3 and Section 2.4.

15.4.3.3 Horizontal bracing

There are two types of horizontal bracing used in multi-storey steel frames:

- Concrete slabs forming diaphragms
- Triangulated bracing

The most effective form of floor diaphragm is metal decking permanent formwork fixed to the steelwork with through-the-deck welded shear studs and with the deck filled with *in situ* concrete (**Figure 15.28**).

Careful consideration is needed with the design of a horizontal diaphragm using PC units supported on steelwork. This is due to the low coefficient of friction between the steel beams and the concrete units and therefore it is important that a positive connection is achieved between the steelwork and the units. If the units are supported on shelf angles then the gap between the units and steelwork requires to be fully grouted. Where PC units are supported on the top flange of the steel beams then shear keys or angle framing may need to be welded to the steelwork to provide the necessary connection and restraint.

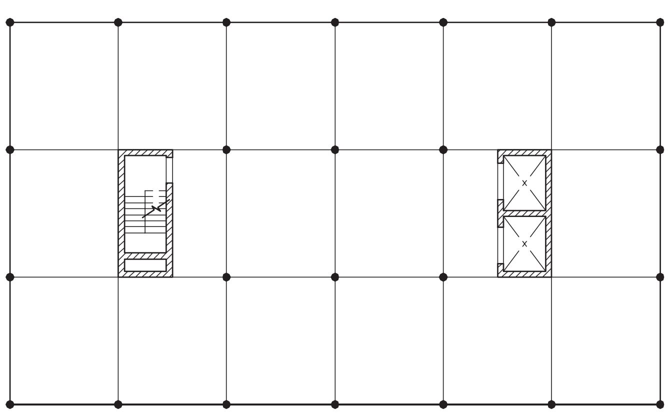


Figure 15.27 Steel framed building with RC stair core and lift shaft providing stability

Where the floor slab is not adequate to act as a diaphragm or the floor slab is interrupted for some reason, for example, because of items of plant or services are penetrating the floor, then horizontal bracing in the form of triangulated floor bracing should be provided to transfer the lateral loads to the vertical bracing (**Figure 15.29**).

15.4.4 Reinforced concrete construction

The braced bays can take the following forms (refer to **Figure 15.30**):

- Concrete core walls – lift shafts, stair wells or service cores
- Concrete shear walls

The shear walls or core walls are reliant upon the floors acting as stiff diaphragms. If the floors are constructed using *in situ* reinforced concrete then a stiff diaphragm can be assumed and no further checks are required. However, if PC units are used then careful consideration of the load path is required (as described above in Section 15.4.2).

15.4.4.1 Design of concrete shear wall

The design axial load on the shear wall is determined assuming that any beams or slabs supported by the wall are simply supported. The design in-plane lateral load is then calculated and this is the sum of two components; the horizontal wind load plus the lateral load due to geometrical imperfections (*equivalent horizontal force* – EHF (refer to Sections 15.2.3 and 15.2.4)).

The shear wall is designed as a cantilever member resisting the axial and lateral loads using the appropriate partial safety factors. The maximum extreme stress in the wall is determined from the following expression:

$$f_{ct} = \frac{N}{lh} \pm \frac{6M}{hl^2} \text{ MPa}$$

Where
N = design ultimate axial load (N)
M = design ultimate in-plane moment (Nmm)
ℓ = length of wall (mm)
h = width of wall (mm)

This stress should then be used together with any transverse bending to calculate the area of reinforcement required.

The shear walls should generally have a minimum thickness of not less than 150 mm to facilitate concreting.

15.5 Precast concrete framed buildings

Precast concrete framed buildings are commonly used for low- and high-rise office, residential, warehouse and industrial type buildings. There are often significant advantages to be had by prefabrication of elements off-site where the manufacture of units can take place inside a casting shop away from inclement weather and where high tolerances in casting and in quality control can be achieved (**Figure 15.31**).

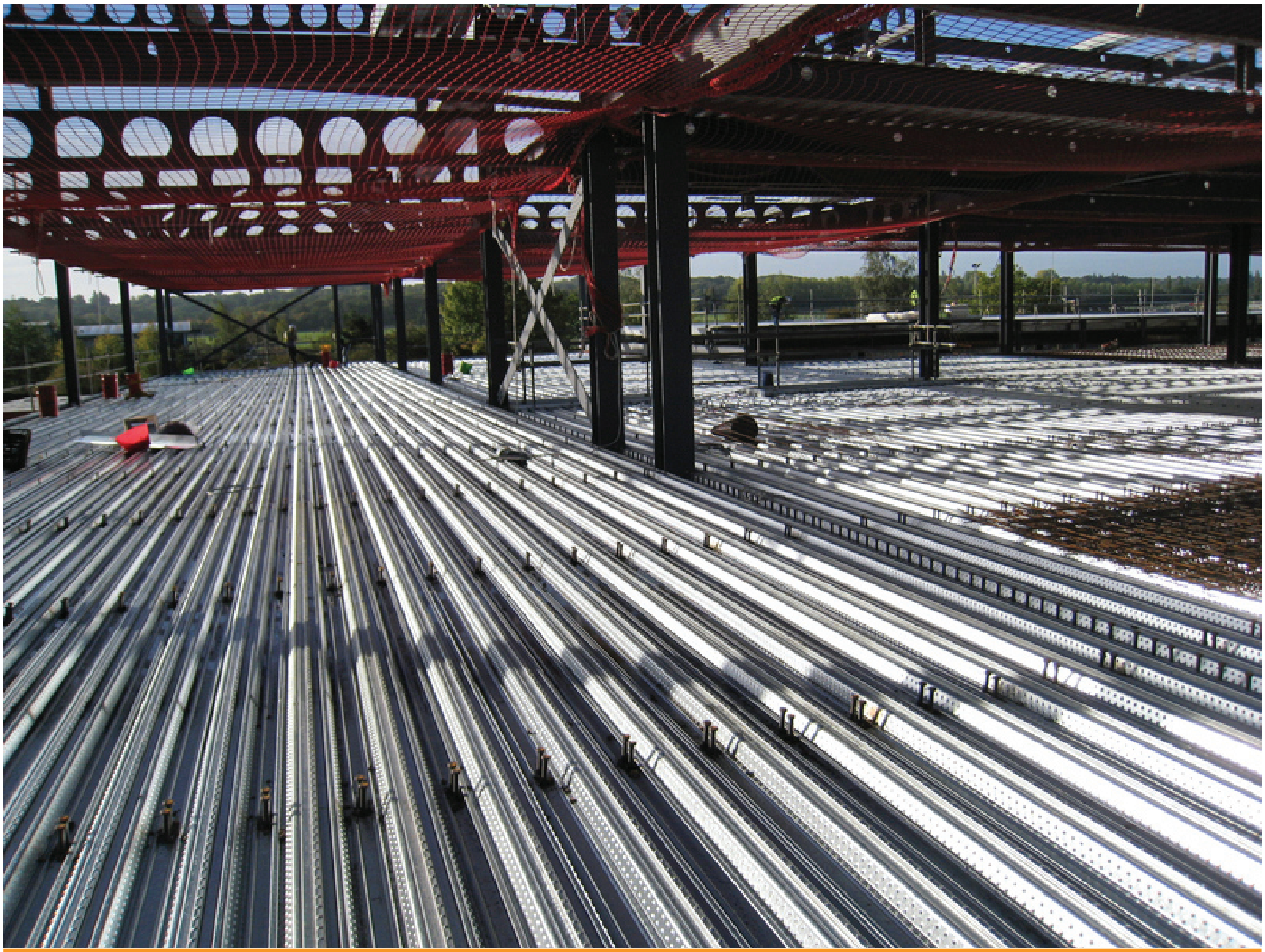


Figure 15.28 Metal decking with through-the-deck welded shear studs. Courtesy of Prodeck-Fixing Limited

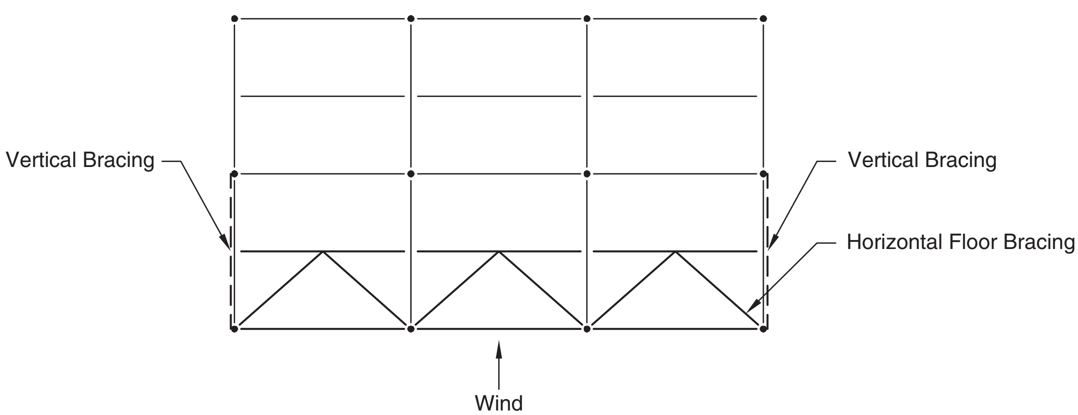


Figure 15.29 Horizontal floor bracing connected to vertical bracing

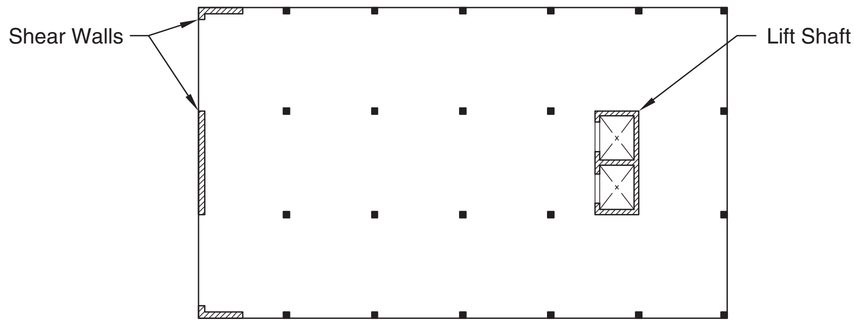


Figure 15.30 RC framed building with lift shaft and shear walls providing stability

15.5.1 Stability of precast concrete framed buildings

The stability of precast concrete framed buildings is achieved in the same way as the other forms of construction previously described, notably floor diaphragms and shear walls (refer to Sections 15.4.1 and 15.4.2 above).

15.5.2 Design of precast concrete shear walls

The shear walls are normally designed as vertical cantilevers with the in-plane stiffness resisting overturning and sliding (refer to Section 15.4.4 above). These walls of course carry a proportion of the vertical load from the floors, roof and walls above. The shear walls can be readily constructed using precast concrete panels (**Figure 15.32**). It can be seen that continuity reinforcement and *in situ* strips of concrete are required to complete the shear wall.

It is noted that masonry infill panels should not generally be used to provide building stability since there is always the possibility that such walls could be removed at some later stage.

15.6 Further stability requirements

Some further stability requirements for various types of structure are discussed below.

15.6.1 Heavy industrial steelwork

Stability design for heavy industrial structures often requires special consideration due to three factors:

- Additional lateral or dynamic loads from items of mechanical plant.
- Sometimes extremely heavy mass at high level.
- Large floor penetrations or discontinuous floor to accommodate plant.

15.6.1.1 Floor construction

Floor construction in an industrial plant typically consists of one or more of the following types:

(a) *In situ* reinforced concrete

(b) Pre-cast concrete slabs with a structural topping

(c) Durbar steel plate/chequer plate

(d) Open grid steel flooring

Floor types (a) and (b) are suitable to act as horizontal diaphragms to transit the lateral loads to the vertical bracing. The design should be sufficiently flexible to allow for additional floor penetrations which may be added at a late stage. The PC slabs will typically require a structural topping 75–100 mm thick comprising small aggregate structural concrete with a continuous layer of fabric.

Floors types (c) and (d) are usually chosen for economy and where future access to plant, pipework or services is likely to be required. This flooring has only nominal shear stiffness and additionally it is often required to be removable hence this floor type is unsuitable to act as a horizontal diaphragm.

Where large openings or plant penetrations occur in a slab which is acting as a floor diaphragm, additional plan bracing must be incorporated in the floor structure to transfer the shear and bending across the opening. If design of the floor structure permits, it is preferable to separate the lateral load resisting system from the plant supports since this is likely to reduce the effect of late alterations.

15.6.1.2 Vertical bracing

Vertical bracing for industrial structures is usually tension only X bracing. The bracing members can be back-to-back angles, channels, hollow sections or UC sections depending on the loads involved and length of the bracing spar. Where X bracing cannot be accommodated then N or K bracing can be used depending upon restrictions imposed by the plant.

The vertical bracing bays are usually arranged at the extremities of the steel framing as previously described. However, the restraint effect of the bracing can cause problems due to expansion or contraction of the steel frame. With industrial structures the bracing can in some instances be extremely substantial imposing high restraint forces on the vertical bracing. In these situations it can be preferable to locate bracing bays near the centre of the structural frame and thereby avoid the thermal effects on the bracing. Erection of the steel frame would therefore commence at the centre of the framing and work outwards (**Figure 15.33**).

Figure 15.31 Cross walls in precast concrete residential building. Courtesy of GPS Precast Concrete Specialists

15.6.1.3 Gantry cranes

Gantry cranes installed within industrial buildings and structures require the gantry rails to be accurately set out and properly tracked. If structural components are fabricated and erected inaccurately then this could lead to crabbing of the travelling crane which could ultimately cause the crane to jam when installed. This in turn could impose large longitudinal forces on the structure causing possible over-stressing of bolted or welded bracing connections.

15.6.2 Construction below ground

Stability of basement and other types of construction below ground can be affected by various actions (see below) that can be critical both in the final design and during the construction stage, effecting temporary works design.

15.6.2.1 Actions to be considered

- Earth pressure – It may be appropriate to design for earth pressure at rest which can be two to three times the active pressure. Similarly if the retained earth is sloping towards the basement structure then the lateral loads can be up to a factor of three times the active pressure.
- Water pressure – Consideration should be given to possible flood conditions both during construction and in the final state. When considering buoyancy, ground water should be taken to be at least ¾ of the depth of the basement. This applies in the UK but in some other countries a full basement depth of water may be appropriate for design, where flood conditions could be experienced during basement construction.
- Surcharge from adjacent access roads or highways.
- Surcharge from adjacent buildings.
- Surcharge from plant loading such as cranes adjacent to excavation.
- Effect of groundwater lowering.

The effect of basement construction on adjacent buildings and services should also be considered.

15.6.3 Stability of buildings during construction

It must be emphasised to builders and contractors the need to adequately brace buildings and structures, during all stages of construction, to ensure they are stable. There are numerous documented incidents where buildings and structures have partially or totally collapsed during the construction process.

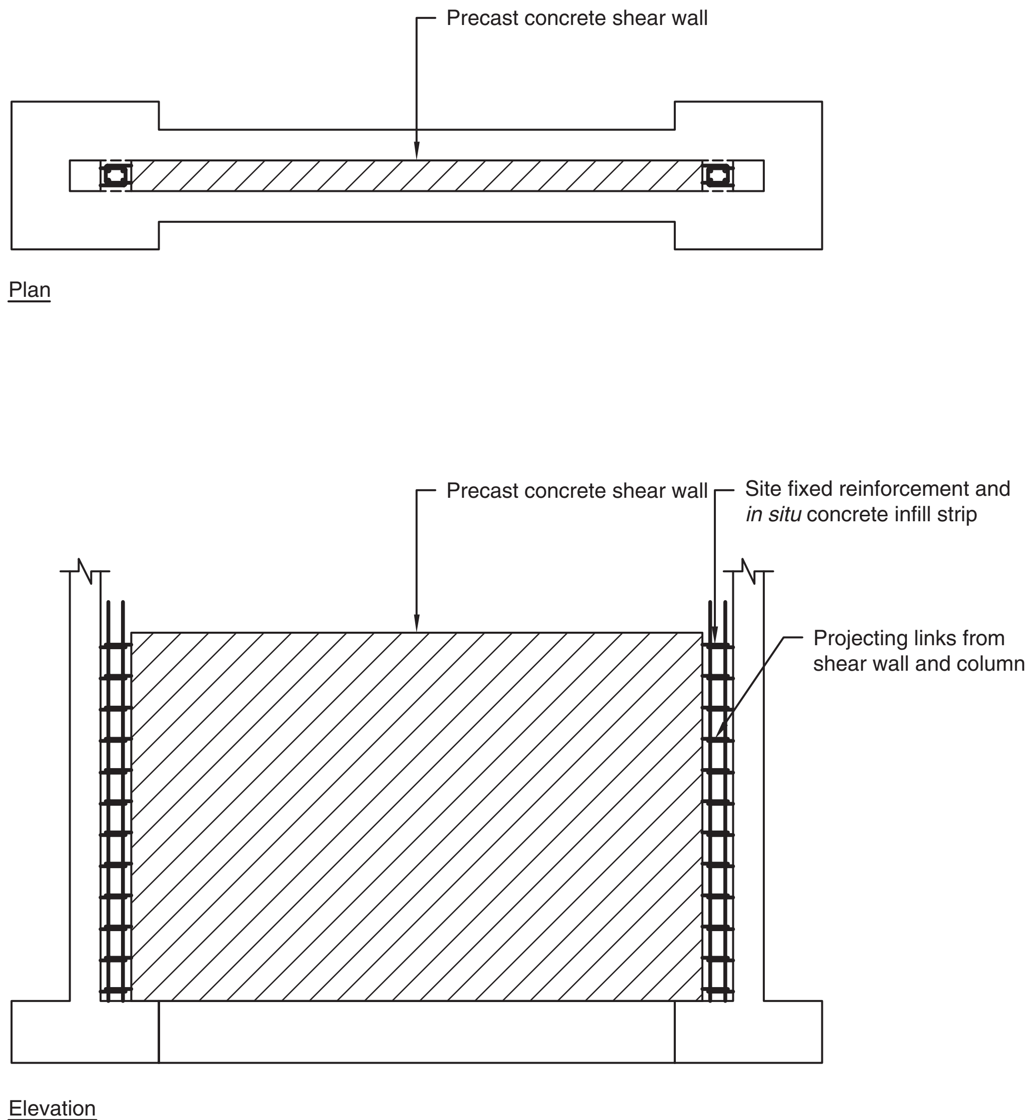


Figure 15.32 Precast concrete shear wall

These collapses have sometimes resulted in death or injury to workers and other people in the vicinity; in addition to the damage to property and the significant financial cost to the builder from the clean-up and rectification works.

15.6.3.1 Lack of adequate lateral support

Building Control and Building Regulations ensure as far as possible that the completed building has been designed to have adequate strength, stiffness and stability during its life; when used for the intended purpose. However, a partially completed erected structure may behave in quite a different manner from that of the completed structure.

Although the designed stability of the completed building may be satisfactory, examination of construction collapses have generally revealed that the builder had not adequately addressed the building's stability during the different stages of the construction process.

Collapses have often been caused by a lack of adequate lateral support, due to the use of 'pinned' connections between structural elements or a reliance on a permanent component of the structure, which had not been installed and temporary bracing had not been provided.

15.6.3.2 System of safe working

To prevent these types of structural collapses a system of safe working should be adopted on site to ensure the building or structure is stabilised and adequately braced against live and dead loads, including lateral loads during all stages of construction.

The system of safe working to maintain stability should include the following:

- The main contractor should appoint an engineer to check the proposed construction sequence and design appropriate temporary bracings or supports. The engineer should be able to clearly define the load paths within the building such that the lateral loads will be safely transmitted to the foundations. This may require input from a structural engineer.
- For each stage of construction, stability and bracing requirements should be documented, either in the work method statement or on the building plans.
- Provision and timing of installation and removal of temporary bracing should be stated.
- Stability and bracing arrangements should be reviewed when structural changes are made to the building.
- Existing bracing, if used, should be checked to ensure the continued robustness of the whole building.
- Ensure that all temporary bracing is installed correctly and is maintained in a serviceable condition until its use is no longer required.

15.6.3.3 Other considerations

The following should also be considered for their effects on the stability of the temporary or permanent works:

- Partial cladding that could affect the magnitude and distribution of wind load.
- Stacking of building materials on temporary works or on completed permanent works such that stability of the partially completed permanent work could be affected.
- Stacking of demolition materials on floors or against vertical walls that are not so designed.

Figure 15.33 Power station steelwork, braced tower. Courtesy of Bourne Steel

15.6.4 Stability of temporary demountable structures

Temporary demountable structures are used at public functions and sporting events. Typically, they include temporary grandstands, platforms, towers and masts to support video screens and loudspeakers. Temporary structures can be subjected to considerable lateral actions due to wind and additionally in some cases, crowd loading causing vertical and horizontal dynamic loads.

15.6.4.1 Design procedure

Each temporary structure, whether a proprietary design or custom-built structure, requires a set of design calculations to be prepared for the superstructure and foundations. These calculations should be prepared by an engineer and should be subject to an independent check preferably by a chartered engineer with appropriate experience. In some instances, these calculations may need to be submitted to the local authority Building Control.

15.6.4.2 Foundations

The supports for temporary structures typically consist of steel posts with a base plate supported on timber spreaders at ground level. Some type of ground investigation needs to be carried out to determine an allowable ground-bearing pressure for foundation design. This should take the form of a desk study, walk-over visual inspection, probing the ground and/or trial pits. It is imperative that any soft spots or poor ground are discovered and replaced with compacted granular fill. Any differential settlement of the foundations can be critical for tall structures.

Whilst the superstructure can be designed for strength and overall stability, the ground conditions may vary significantly from site to site and therefore individually designed ground support systems may be required.

15.6.4.3 Design considerations

- Temporary structures should be designed for static loading and dynamic effects where appropriate.
- Wind loading should be in accordance with Eurocode 1 – Parts 1–4: Wind. For temporary structures account can be taken of seasonal effects and short duration of exposure.
- The need to consider uplift forces where tension fabric structures are proposed, for example, screw piles for foundations.
- Notional loads can take account of spectator movement and geometrical misalignment in the frame. Notional loads should be considered in conjunction with wind loading.
- Crowd overloading should be considered if appropriate.
- Horizontal sway of the frame due the combined effect of dead, imposed, wind and notional loading should be restricted to height/300.
- For those structures such as temporary grandstands, which can be subject to synchronised crowd movement, dynamic effects need to be taken into account in accordance with BS 6399 Appendix A. In some circumstances a rigorous dynamic analysis will be necessary.

- Limitations on the out-of-plumb tolerance of the structure should be determined in the calculations.
- Stability should be checked to ensure:

  Restoring moment > 1.5 × overturning moment using unfactored loads.

For more detailed guidance on this topic reference should be made to IStructE (2007).

15.6.5 Stability of aluminium structures

Aluminium alloy has a density about 1/3 that of steel. Typically the weight of an aluminium structure is about 1/2 that of a similar steel structure. The lower density makes the material obviously suitable for lightweight structures, particularly lightweight roof structures such as space decks.

Aluminium has the characteristic of good formability, so it can easily be extruded. Aluminium can also be cast, drawn and machined. It is therefore able to be produced in a wider range of sections than steel.

A further property of importance to the structural engineer is the modulus of elasticity of aluminium alloy which is about 1/3 that of steel. Both the buckling capacity and deflection of members are proportional to the 'E' value. Considering buckling, therefore, a larger aluminium member will be required compared to an equivalent steel member for the same load. It is noted that the buckling analysis of aluminium members is more complex than for steel. Regarding deflections these will be greater in an aluminium member compared to a similar steel member. The lower buckling capacity of aluminium members clearly affects the design of stability bracing.

15.7 Conclusions

In this chapter some common forms of construction have been described together with typical means of achieving building stability. In a review of this type, not all forms of construction or types of stability systems can be included. However, whether structural steelwork, *in situ* concrete, precast concrete, structural masonry or timber, permanent or temporary works the same basic principles exist for achieving building stability.

The stability design process involves the proper assessment of the overturning or lateral actions, and the transmission of those forces through the building to the foundations. At each stage of the load transmission, adequate factors of safety should be provided and ultimately the foundations should be designed to safely resist the overturning actions. As stated earlier, the load paths should be as direct as possible and as emphasised, it is most important that the engineer has a clear understanding of those load paths.

15.8 References

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15.8.2 Further reading

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15.8.3 Useful websites

Steel Construction Institute (SCI) – www.steel-sci.org
Concrete Centre – www.concretecentre.com
Timber Research and Development Association (TRADA) – www.trada.co.uk
Brick Development Association (BDA) – www.brick.org.uk

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CONTENTS

Chapter 16

Movement and tolerances

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All construction activities require the assembly of often complex parts, some of which are pre-fabricated in the field and others which are constructed on site, to produce a one-of-a-kind product. The structural engineer designs the assembly that connects these parts together, so a well-developed understanding of the assembly process and the potential for material dimensional deviation is a prerequisite skill. In this context, this chapter aims to provide an overview of movement and tolerances issues inasmuch as they affect the work of the practising structural engineer. It is not intended as a comprehensive reference document, but provides sufficient introduction and further references on the subject for those that require it.

16.1 Introduction

All construction activities require a complex series of parts to be arranged to create what is often a unique product. The designer of the framework that binds these complex parts together must therefore have a well-developed understanding of the assembly process and the potential for material dimensional deviation. In this context, 'tolerance' can be defined as 'the permissible range of variation in a dimension of an object, or the permissible variation of an object or objects in some other characteristic such as hardness, weight, or quantity'. Thereafter, this understanding of tolerance must be coupled with recognition of how an assembly of complex parts might dimensionally change with time, or 'move'.

The structural engineer is the designer of the skeletal frame that holds together complex buildings and pieces of infrastructure, so it is fundamental that they have a well-developed understanding of construction tolerance and expected movement. Buildings, infrastructure and their components will seldom have dimensions that match those shown in the construction drawings and specifications. A realistic view of the dimensional variability inherent in the construction process is therefore required so that appropriate details and assemblies can be achieved in practice without unnecessary tolerance constraints or construction difficulties. Such problems can lead to unsatisfactory contractor performance, unsightliness in the finished product and poor in-service performance, all of which can be difficult and costly to overcome.

The aim of this chapter is not to act as a single-source reference document for structural engineering-related movements and tolerances minutiae. Such a task would warrant a book in its own right and noteworthy design code guidance and best-practice reference documents already exist. Instead, this chapter aims to provide an overview of movement and tolerances issues inasmuch as they affect the work of the practising structural engineer, providing references for further reading on the subject for those that require it.

Given ever-increasing professional globalisation and the gradual convergence of design practices, the author has sought to reference both European and American design codes and best-practice documents as a first point of call. This is a reflection of the author's linguistic limitations and experience. In practice, the onus remains on designers to fully familiarise themselves with local codes of practice, material standards and construction norms, making sure that due allowance is made within their documentation for inevitable local differences in approaches.

16.2 Tolerances

16.2.1 The rationale for construction tolerances

It is important to define the distinct reasons for construction tolerances so that those that apply in any given case can be correctly specified, monitored and if need be corrected (see **Table 16.1**).

16.2.2 Definition of deviations and tolerances

The means for specifying and monitoring tolerance requires a range of permissible deviations, tolerances and ranges to be set (see **Table 16.2**).

16.2.3 Understanding best-practice and real-world limitations

Multi-disciplinary project design requires designers to have an understanding of how things are made, how they are assembled and thereafter how they will fit together over time. An attempt to capture this was made with BS 5606:1990 *Guide to Accuracy in Building* (BSI, 1990). This is a guidance document for designers and it covers buildings generally. Its primary goal is to explain tolerances issues so that due allowance is made in design documentation for reasonable field adjustments. Importantly it does not call for unattainable levels of

Material in-service behaviour	Materials change shape with time according to load and environmental conditions, affecting how elements fit together
Material deviation	Structural design codes assume material performance tolerances in terms of strength, elasticity, ductility and other properties. Deviation beyond these could impair design integrity and serviceability
Structural design	Structural design codes assume dimensional tolerances in terms of element size and shape, and construction assembly. Deviation beyond these could impair design integrity
Coordination	A complete construction is a sum of its parts, requiring non-structural elements such as cladding, building services, lifts and finishes to fit together without clashes or undue distress
In-service performance	The structure must remain sufficiently straight and true so that its function, or the function of other constituent non-structural parts such as cladding, lifts or building services, is not impaired
Visual appearance	Limits on verticality, straightness, flatness and alignment may be required, particularly with respect to architectural finishes
Boundaries/ adjacencies	Site boundaries and other adjacencies such as buried infrastructure or adjacent tall buildings may necessitate design movement and tolerance limitations
Moving parts clearances	Moving parts such as travelling cranes, rail tracks, elevators, mechanical car-parking and large doors may require strict construction and in-service tolerances to ensure snag-free use

Table 16.1 Rationale for construction tolerances

Deviation	Difference between a specified value and that actually measured, always expressed vectorially (i.e. +/– value)
Permissible deviation	Vectorial limit (i.e. +/– value) specified for a particular deviation
Tolerance range	Sum of absolute values of the permissible deviations either side of a specified value
Tolerance limit	Permissible deviations each side of a specified value (e.g. +/–10.0 mm or +10.0 mm/–5.0 mm

Table 16.2 Definition of deviations and tolerances

construction accuracy and its thoroughness means that it can be used as a reference for most projects.

It is rarely economic or necessary to achieve extreme levels of accuracy, so tolerances should never be specified closer than required. These should be considered at the start of a project to ensure that:

- design details are practical, easy to fit together and can accommodate the individual build-up of dimensional variables;
- design details reflect the needs of the structural design;

Standard tolerances	These are usually necessary for all buildings, varying according to material type. They are normally codified or based on industry standard reference documents, making them widely understood and standard for most structural assemblies. Codification also means they form an integral part of the assumptions on structural design compliance
Particular tolerances	These are tighter than standard tolerances and usually only apply only to certain components or dimensions. Their use is normally governed by localised reasons of fit-up, interference and clearance, or to respect certain boundary restrictions
Special tolerances	These are tighter than standard tolerances and usually apply to a complete portion of structure and sometimes a project. Their use may be required in special cases for reasons of serviceability or architectural appearance, and sometimes for structural reasons (i.e. dynamic or cyclic loading or fatigue). Factory pre-fabrication and/or special assembly requirements (i.e. reuse or speed of assembly) can both facilitate and require this higher level of control

Table 16.3 Tolerance classifications

- each project participant understands how their respective details will fit together;
- compliance with the requested tolerances can be monitored during construction.

On the proviso that this advice is followed, it should be feasible to specify standard tolerances such as those presented in BS 5606 or other industry best-practice documents unless they conflict with an overall need for greater structural, architectural or other accuracy. Thereafter design drawings and specifications can be prepared with details that clearly call up the need for increased accuracy when it is appropriate. In practice, these are classified as noted in **Table 16.3**.

This classification system, with its emphasis on practicality and the use of standard tolerances wherever possible, is reflected in both Eurocode and American design approaches.

The Eurocode has taken this a step further for steel, aluminium and precast concrete design with the definition of a series of Execution Classes (EXC 1 to 4) that define the level of quality assurance and workmanship applied to different structures and their components. This is because these elements tend to be fabricated off-site where better workmanship and quality control can be achieved allowing the designer to fine-tune the design and construction. This is particularly important where fatigue or architectural quality control issues need careful attention. The decision to use the higher classifications must be made in conjunction with the client, as it requires the contractor to have factory production control (FPC) systems in place similar to ISO 9001 that confirm their ability to work against the chosen Execution Class. This usually implies an increased construction cost.

The designer should always be mindful that building codes are usually guidance documents and the engineer always retains the responsibility of determining the appropriate design criteria, applicable codes and best practices for a particular project. This is engineering judgement.

16.2.4 Special tolerance conditions

In some cases it may be necessary to assess the combined variability effects of the differing parts that constitute a fabricated element or component to make sure that they will fit together. In more sophisticated or architecturally challenging structures this may extend to construction of a full-scale mock-up to test the tolerance provisions at critical interfaces.

16.2.5 Best-practice guidance on movements and tolerances

As already noted, BS 5606 provides designers with comprehensive guidance on the issues of movement and tolerances, whilst the *Handbook of Construction Tolerances* (Ballast, 2007) provides a very comprehensive single-source reference document based on North American construction practice. These documents provide a starting point for all designers, irrespective of discipline. Further material and discipline-specific movement and tolerances codes and guidance documents can then be used in accordance with project typology. See Section 16.10 for further references.

16.3 Material behaviour and movement under applied load

Inherent material variability and in-service behaviour will have an effect on construction tolerances. This section describes these effects, with particular emphasis on how different materials respond to environmental conditions and loading so that in-service material behaviour is understood. See Chapter 10: *Loading*, for more information.

16.3.1 Inherent material standard deviation

Material properties and fabricated elements all differ, making it impossible to produce identical products. Standard deviation is used as the measure of this variability, calculated using statistical and probability theories. If a small data sample set is available, the population standard deviation can be estimated using a modified quantity called the sample standard deviation. All design codes, material references and tolerance allowances account for this inherent variability.

16.3.2 Isotropic versus anisotropic movements

Material responses to environmental and applied loads can be direction-dependent. Isotropic behaviour is identical in all directions, whilst anisotropic behaviour depends on the orientation of internal fibres. The assumption that a material has isotropic properties is often a good starting point for primary structural materials such as steel, concrete and masonry, but possible anisotropic response to loads and environmental effects must be reviewed as part of the structural design process. Obvious anisotropic behaviour is exhibited in both natural and engineered timber products, membranes and structural composites since their fibre orientation fundamentally affects their response to all loading.

16.3.3 Material thermal responses

Most materials expand when heated and contract when cooled, and those that show the opposite behaviour have uses that are currently restricted to scientific research laboratories. All commonly used construction materials fall into the former category, but an important exception to this rule is water. Unlike most substances, its solid form is less dense than its liquid phase. A block of most elements will sink in its own liquid but a block of ice floats in liquid water. Therefore, the structural design of elements exposed to the environmental effects of snow and ice build-up (e.g. an external structural wall, roof or a roof top water tank) must account for this expansive effect.

The term *linear coefficient of thermal expansion* (α) is used to describe how much a material will expand for each degree of temperature increase, as given by the formula:

$$\alpha \, dt = dl/L$$

where:

- dl = the change in length of material in the direction being measured
- L = overall length of material in the direction being measured
- dT = the change in temperature over which dl is measured

The ratio is dimensionless, and is normally quoted in parts per million per °C rise in temperature. Its related volume coefficient of thermal expansion is rarely used in common structural engineering problems. It is standard practice to equate many self-straining forces (e.g. those arising from differential settlements of foundations, restrained dimensional changes due to temperature, moisture, shrinkage, creep and similar effects) to an equivalent temperature load on a structure, as the analysis can be idealised in a straightforward manner.

An additional effect of material thermal response can come from manufacturing or placing processes like smelting or oven baking (e.g. steel, aluminium, glass, masonry and composites manufacture). These create elements that are warped and not perfectly smooth as the constituent parts of the elements cool at different rates. The placing of wet concrete and its subsequent hydration is an exothermic process. This initially heats the concrete after which it cools down to match ambient temperatures, mimicking the effect seen in other materials as they cool.

The variation in cooling rate experienced by these materials creates internal variations in their residual shrinkage, both along their element lengths and through their cross-sections, creating internal residual stresses. Structural engineering design codes acknowledge these issues and deal with them in material-specific ways, but in the context of construction tolerances it is important that due consideration be given to the fact that materials are rarely perfect in shape and that the uncontrolled application of heat can change material properties.

In long or tall structures diurnal temperature variations can influence the construction process, requiring all setting-out to be undertaken in the early morning before the structure is warmed by the midday sun.

16.3.4 Material moisture responses

Internal changes in moisture can affect some commonly used materials in respect to movement and tolerances; particularly concrete, masonry and timber. This causes irreversible shrinkage in most materials, apart from timber which will re-expand as its moisture content increases.

In some cases moisture ingress can facilitate material degradation or corrosion; expansive ice formation in wet masonry and concrete can lead to surface cracking; likewise steel corrosion is usually an expansive reaction that can crack adjacent concrete cover and brittle facades. Structural laminated glass in contact with water over extended periods can degrade – caused by interlayer de-bonding from the glass surface – affecting its long-term performance. It is beyond the remit of this chapter to cover architectural and structural detailing, other than to note that insufficient environmental protection of structural elements can lead to deleterious movement and tolerances impacts on a completed building.

Other indirect and potentially detrimental moisture effects include those associated with foundation design. These can fundamentally affect soil load capacity and structural settlement.

16.3.5 Deflection under load

All commonly used construction materials deform under loading; this can be elastic (i.e. reversible), plastic (i.e. irreversible) and in some cases time-dependent (i.e. it can increase with time or 'creep'). It can be axial, torsional, rotational or a combination of all three.

16.3.6 Elastic deflection

Elastic deflection is reversible; the object regains its original shape once the applied loads have been removed. All commonly used construction materials exhibit varying degrees of this property.

Hooke's law is used to determine linear elastic deformation:

$$\sigma = E\varepsilon$$

where:

σ = applied stress

E = material constant termed the Young's modulus

ε = resulting strain

The relationship only holds in the elastic range that ends when the material reaches its yield strength after which deformation is plastic. Many common construction materials, including masonry, concrete, timber, cast iron, membranes and glass, can exhibit nonlinear responses when loaded in the elastic range.

16.3.7 Plastic deflection

Plastic deformation is irreversible; an element loaded in the plastic deformation range will only regain the proportion of its shape equivalent to its initial elastic deformation, with the remainder of its plastic deflection being non-recoverable. Hard or brittle materials such as masonry, concrete, composites and membranes have minimal plastic deformation ranges, whereas steel and timber have larger ones. This explains why steel and timber perform well under seismic loading, as they can deform plastically to absorb the energy unleashed on the building by a seismic event.

Most soils show highly nonlinear plastic behaviour, although some cohesive soils can regain some of their shape if the applied loads are reduced.

16.3.8 Material creep

Creep is the phenomenon exhibited in a solid material when it slowly and permanently deforms over time under applied stresses that are below its yield strength. Creep increases with temperature, and it is more severe in materials that are subjected to constant heat, or those near melting point (e.g. steel in a fire situation when it permanently buckles).

The creep deformation rate is a function of inherent material properties, the applied stress, the duration of load, the exposure temperature and in some materials the exposure humidity. Whilst creep does not constitute a material failure mode, excessive deformations can mean that a component or items fixed to it can no longer perform their function (e.g. if a reinforced concrete (RC) column deflects under creep load and cracks the floor slab and finishes attached to it). Paradoxically, creep can sometimes be beneficial where it relieves tensile stresses that might otherwise lead to cracking (e.g. in an RC floor slab).

16.3.9 Self-weight and applied loads

All structures are designed to support a combination of their own self-weight, superimposed dead loads, live loads, environmental loads and induced settlements. Self-weight and superimposed dead loads are deemed to be permanent and easily quantifiable, whilst live (or imposed) and environmental loads can be temporary or transient, calculated using probabilistic analysis of their likelihood and size. This definition of permanence is used to set up the load factors used in standard load combinations; permanent loads and quantifiable loads such as self-weight, superimposed dead load, flooding and self-straining forces such as temperature and shrinkage attract lower factors; less quantifiable live and environmental loads such as floor, snow, wind and seismic loads attract higher factors to compensate for their higher variability.

This differentiation must be quantified to determine the applied loading on a structure, and thereafter its elastic and plastic settlements. It is also needed to determine the creep settlement, as the structural engineer will need to use a breakdown of permanent and transient loads to determine the most-likely applied stresses and their associated creep deflections.

Determination of settlement according to loading type means that its use in a movement and tolerances assessment can be time dependent. Limit-state structural analysis techniques assume that all loads are applied simultaneously, when in reality the loads are built up in a piecemeal fashion. Time-dependent load assessment therefore allows designers to focus on specific tolerance issues, and it is standard practice to set deflection limits according to loading type (e.g. self-weight, superimposed dead and live loads). These are then set against the potential effect of the deflection, allowing more flexibility and greater deflections if it is non-detrimental (e.g. a long span roof with non-brittle finishes will have a greater live load deflection allowance than a roof with brittle finishes that might crack).

Time dependency is also important when looking at construction sequences, as deflections and settlement can be built out by pre-setting structural elements or their formwork to account for their expected future movements (e.g. pre-cambering elements to allow for horizontal deflections, or increasing element lengths to allow for vertical axial shortening). This is especially useful and most commonly witnessed in long-span structures, high-rise construction and complex steelwork erection.

16.3.10 Snow load

Snow loading is a feature of colder climates, and while the load duration varies it is unlike a live load since it is considered to be uniformly applied at a given time. Its magnitude is highly dependent on local weather pattern, terrain and latitude, and roof geometry plays an important role in assessing snow drift (i.e. concentrated load build-up in one area). Codified design snow loads are statistically determined from historical records but it is not uncommon to have unusual events when these design values are exceeded. Snow load regularly exceeds roof live load allowances, and freezing and thawing of snow can create ice and wet snow which are heavier than snow.

Local regulations sometimes require building owners and designers to consider active load control methods such as sloped roofs or mechanical roof snow clearance to mitigate these effects, although in areas with weak local enforcement tendencies this latter method can actually be banned as a design load and deflection mitigation strategy.

16.3.11 Seismic load

Earthquakes are the result of sudden energy releases due to slippage between tectonic plates along a geological fault in the earth's crust. They are mostly natural but drilling, mining, dam construction and deliberate injection of water into faults can trigger events. These create various types of ground excitation as the seismic waves propagate, and any structure located along this propagation path will be subject to ground motion that imposes vertical and horizontal forces on the structure. Codified design practice in the field is well developed in terms of standard structures, and if followed the structural engineer will be able to quantify these expected in-service deflections. Highly irregular structures tend to be discouraged and penalised by these design codes as their behaviour and torsional response is harder to predict and generally under-studied.

Buildings and structures are classified according to their risk profile in terms of potential loss of human life, potential for environmental damage and their role in national and self-defence (e.g. power stations, hospitals and other emergency preparedness facilities are given the highest risk profile, whilst agricultural buildings are given the lowest). This classification will determine the expected level of structural damage, non-structural damage to fixtures and fittings, and loss of life in a seismic event. The higher the risk, the more onerous the structural design criteria and loading, structural detailing and restrictions on building design.

In the context of structural movements and tolerances, this will need to be well communicated to the wider project and client team as the higher-risk categories can be restrictive in terms of building layouts and their architectural detailing. Seismic-resistant structures need to be ductile in a seismic event, capable of deflecting and absorbing energy as the ground shakes the building. High-rise towers with low frequencies of excitation, and flexible steel, timber and RC frame structures perform better in this respect. Short buildings with rigid masonry or RC bearing wall systems perform worst as their stiffness does not facilitate energy dissipation. This means that non-structural fixtures and their connections must be capable of resisting relatively large loads and deflections which will have an impact on their cost and architectural appearance.

16.3.12 Wind load

Differential surface heating of the earth creates areas of high and low air pressure, creating wind pressures as these warmer and colder areas seek to balance. Local wind pressure characteristics are a function of approaching wind pressure, the structural geometry under consideration, and the geometry and proximity of the structures up and downwind. These pressures fluctuate highly, and in certain cases can result in fatigue damage to structures if they become dynamically excited. Codified design practice is well developed in terms of standard structures, and if this is followed the structural engineer will be able to quantify these expected in-service deflections.

Exceptions to codified practice are highly irregular, lightweight or potentially dynamic structures, large bespoke structures such as bridges and chimneys, and high-rise towers. In these fields experimental wind tunnel tests are usually undertaken to better quantify loads, and their use in facade engineering is becoming an increasingly common means of value engineering expensive curtain wall systems. On occasion wind tunnels are also used to fine-tune physical building or structural shapes for improved wind load response. This has the twin goal of reducing wind loads by improving the aerodynamic wind profile of the structure, and to eliminate any potential risk of vortex shedding.

When a fluid flows around an object it creates alternating low-pressure vortices on the downstream side of the object. This is termed vortex shedding, and the object will tend to move towards the lower-pressure zone. If the vortex shedding frequency matches the resonance frequency of the structure it will begin to resonate and its movement can become violently self-sustaining and potentially self-destructive. Simple changes in a building facade or bridge profile, or the addition spoiler fences to a chimney can break up these vortices, reducing wind loads and consequent deflections.

When high-rise buildings or chimneys are constructed close together the wind profile of one can affect that of those adjacent, in some cases increasing their wind loading. In such cases the structural engineer has a duty of care to review this potential problem to see whether it could be problematic or not.

16.3.13 Construction, traffic, crane and other moving loads

Construction, traffic, crane and other types of transient loads differ from standard building loads as they introduce a dynamic load effect which must be accounted for in design. In most cases the applied loads are presented as equivalent static loads that incorporate and factor-up the dynamic loads. This is beyond the intended scope of this chapter, and specialist and/or manufacturer advice is often required if these loads and their settlement impacts are to be correctly assessed.

16.3.14 In-service stress and differential movement

If the in-service stress of adjacent loaded elements differs, their corresponding elastic, plastic and creep deformations will also differ. Therefore any elements that are connected to them will experience a differential movement or settlement. This most often requires review in foundation design, especially if parts of the building have different foundations (e.g. raft and pile foundation), or if parts of the building have differing soil support.

In high-rise towers or complex structures with multiple transfer systems adjacent vertical column or wall elements can be subjected to different levels of in-service stress, especially if some of the elements have been sized to resist transient lateral loads. Such occurrences impose differential settlement on interconnected structural and non-structural elements such as slabs, beams, facades and other finishes.

16.3.15 Material fatigue

Fatigue occurs when a material is subject to cyclic loading and unloading at loads above a specific threshold. Microscopic cracks are created and once they reach a critical size sudden fracture can occur. Material shape has a significantly effect on fatigue life (e.g. square holes or sharp edges and corners), but the most critical factor to control is the cyclical stress. In most standard structures material fatigue studies are rarely required, but structures subject to any form of wind or dynamic excitation must be reviewed. Fatigue design requirements usually require increased element sizes, tighter control of in-service deflections and movements (i.e. excitation) and strict quality control of the manufacturing, assembly and erection process.

16.3.16 Vibration sensitivity

If tolerance is viewed from the perspective of a variation in a characteristic behaviour, then human, building and equipment responses to building behaviour must be considered under serviceability conditions. Sources of vibration include wind and seismic activity, external vibrations from railways, pedestrian excitation and dancing, and mechanical equipment loads.

In the context of building performance, these vibrations may cause structural fatigue or disturbance to sensitive laboratory equipment, or they may impair the user experience through annoyance or even nausea in extreme cases. Vibration and motion perception threshold levels differ significantly depending on location and context; someone walking across a rope bridge would accept structural motion as they walk but it would likely unnerve them if they felt the floor move whilst walking in a low-rise hospital building. In a tall building, users tend to accept some form of movement as it can be contextualised against the building height and high winds outside.

In low-rise buildings with long floor spans, sensitive equipment or proximity to an external source of vibratory nuisance, the structural engineer should aim to isolate the source of vibration wherever possible by means of passive insulation or isolation methods that have been coordinated with the other design disciplines. If this cannot be achieved, it is standard practice to stiffen the structure to minimise these effects in line with codified and best-practice guidance on the subject.

In high-rise buildings wind response is very sensitive to both mass and stiffness, which can be controlled by increasing either or both of these parameters to increase structural damping. However, this conflicts with earthquake design and material optimisation strategies, so careful definition of vibration design criteria is required that accounts for building use and likely human perception and tolerance of the issue (e.g. an office worker is likely to be less sensitive to vibration than someone lying in bed at home in their apartment). There have been many moving room studies on this subject, undertaken with the aim of better quantifying the relationship between human perception and tolerance of multi-direction building excitation. These have resulted in best-practice guidance that can be used to set design vibration criteria against a range of root mean square (RMS) and peak accelerations limits. In vibration sensitive tall or slender structures tuned-mass or viscous dampers allow serviceability deflection objectives to be met in a materially efficient manner.

16.4 In-service performance of structural materials

16.4.1 Steelwork

Structural steel exhibits the following material in-service characteristics, and structural design and calculation of movements should account for these issues:

- applied load deformation is linear elastic up to the yield stress and plastic thereafter;
- response to environmental changes and applied loads is isotropic;
- response to low temperatures (i.e. non-fire) loads is linear elastic;
- non-corrosive response to environmental humidity changes is negligible;
- element corrosion response is expansive;
- creep rate is low and implicitly included in codified stress–strain–temperature formulae;
- high creep rates in fires are dependent on rate of heating, temperature and applied member stress;
- fatigue failure is possible under cyclical loading.

Structural steel has three types of dimensional tolerance:

- manufacturing tolerances; plate thickness, flatness and section dimensions;
- fabrication tolerances; dependent on workshop quality control;
- erection tolerances; dependent on site construction quality control.

European and American codes of practice and material standards are well-developed with respect to manufacturing, fabrication and erection tolerances. In most cases the inherent improbability of all unfavourable extreme deviations occurring together is small, and simple means of on-site adjustment can be incorporated to avoid the cumulative accumulation of deviations. These include packing pieces, slotted holes and threaded rods. See Section 16.10 for further references.

Tighter specification of movements and tolerances may be required in the following cases:

- High-rise tower lift shafts: steelwork verticality and horizontal deflection under wind load need to be carefully controlled in line with lift manufacturer specifications.
- High-rise tower external columns: coordination tolerances with facades are crucial.
- Architecturally exposed structural steel (AESS): high-quality connections and details.
- Fatigue-sensitive connections: quality control of welding and connections.

See Chapter 18: *Steelwork*, for more information on steel.

16.4.2 Reinforced, post-tensioned and precast concrete (light and normal weight)

Structural concrete exhibits the following material in-service characteristics, and structural design and calculation of movements should account for these issues:

- Applied load deformation is elastic and plastic below and above yield stress.
- Response to environmental changes and applied loads is broadly isotropic, although unidirectional cracking is common in incorrectly designed and constructed structures.
- Response to low temperatures (i.e. non-fire) loads is linear elastic.
- Shrinkage, strength gain and cracking are all affected by environmental humidity.
- Lightweight concrete properties are heavily aggregate dependent; for equivalent concrete strengths its elastic properties, compressive and tensile strength, creep properties, durability and fire resistance differ from those of normal concrete.
- Element corrosion and degradation response is expansive.
- Creep is dependent on mix design, environmental conditions and loading, and in non-standard or large elements a key design consideration.

Structural concrete has three types of dimensional tolerance:

- Manufacturing tolerances: constituent materials and mixes are naturally variable.
- Environmental tolerances: dependent on actual mix poured and reinforcement placed.
- Erection tolerances: dependent on site construction quality control.

European and American codes of practice and material standards are well developed with respect to manufacturing, environmental and erection tolerances. In most cases the inherent improbability of all unfavourable extreme deviations occurring together is small, and simple means of on-site adjustment can be incorporated to avoid the cumulative accumulation of deviations. These include packing pieces at non-structural connections, whilst slabs can either be ground or coated with liquid latex to smooth out imperfections. See Section 16.10 for further references.

However, the designer should also be aware of anisotropic effects in RC slabs when concrete expansion or contraction movements are restricted, or when the aspect ratio of panels is excessive. In such cases, cracking can be uni-directional unless appropriate levels of reinforcement are specified.

Tighter specification of movements and tolerances may be required in the following cases:

- High-rise tower lift shaft construction: wall verticality and horizontal deflection under wind load need to be carefully controlled in line with lift manufacturer specifications.
- High-rise construction where floor pre-setting is used to control differential movement.
- Architecturally exposed or unfinished concrete: high-quality panels and joints.

In the case of precast concrete off-site construction can be readily used to achieve tighter tolerances but care is required as they may be too onerous for the actual site conditions.

See Chapter 17: *Design of concrete elements*, for more information on concrete.

16.4.3 Timber

Natural and engineered structural timber exhibits the following material in-service characteristics, and structural design and calculation of movements should account for these issues:

- Applied load deformation is elastic and heavily influenced by grain/fibre direction; brittleness precludes design in the plastic range.
- Response to environmental changes and applied loads is anisotropic.
- Response to low temperatures (i.e. non-fire) loads is linear elastic.
- Orthotropic properties: differing mechanical properties in all three orthogonal directions.
- Hygroscopic properties: shape, size, strength and creep properties are all affected by environmental humidity.
- Creep is dependent on load duration, moisture content and applied load direction in relation to fibres.

Natural and engineered structural timber has four types of dimensional tolerance:

- Source tolerance: constituent materials are naturally variable, with structural performance dependent on type of log cut, timber seasoning and preparation.
- Manufacturing tolerances: dependent on factory quality control.
- Environmental tolerances: dependent on exposure and use of material.
- Erection tolerances: dependent on site construction quality control.

European and American codes of practice and material standards are well developed with respect to the processing, manufacturing, environmental and erection tolerances. In most cases the inherent improbability of all unfavourable extreme deviations occurring together is small, and simple means of on-site adjustment can be incorporated to avoid the cumulative accumulation of deviations. This is especially true with timber, which can easily be site cut to size or levelled with packers to smooth out imperfections. See Section 16.10 for further references.

However, the designer should always be mindful of its anisotropic behaviour and its tendency to absorb and release moisture in accordance with local humidity. Both significantly affect its structural properties and performance, especially in the case of engineered timber products where properties are product-dependent.

Timber is normally kiln dried down to 15–19% moisture content (MC), and sometimes to 8–10% MC using radio waves. This latter technique attracts a cost premium but if stored incorrectly it will re-absorb moisture. Dry timber is usually defined as having an MC no greater than 19%, so drying to values below this means that any drying defects will become apparent prior to use. This MC is measured as a ratio of the weight of the water in the wood relative to its oven-dry weight. Timber can hold more than its dry weight of water (i.e. MC greater than 200%), and MCs greater than 20% (i.e. green timber) make it susceptible to attack by dry rot spores. It is defined as fully saturated when its MC is 28%, as kiln drying makes its cells collapse and it cannot hold more than 28% MC after that. Dry timber is often stamped with the letters S-DRY (i.e. surfaced dried) or KD (i.e. kiln dried).

Potential defects from incorrect drying or moisture re-absorption include wood warping, shakes and splits, honeycombing due to differential MC across a piece, cracks around mechanical connections or opening-up of mitre joints or tenon shoulders. Correctly cured and stored timber products should therefore have fewer problems in a finished building or structure, as the product should hold its installation dimensions. Typical in-service MCs are as follows:

- 15–20% MC for external joinery and structural timber.
- 10–15% MC for internal joinery and furniture in non-humid conditions.
- 8–10% MC for internal joinery in rooms that are continuously heated.

This can be used to clearly coordinate and specify design details in accordance with intended environmental and construction conditions. If need be, the timber can be stored in conditions similar to the intended final condition to allow its moisture content to be equalised prior to fabrication and erection.

Tighter specification of movements and tolerances may be required in the following cases:

- Pre-fabricated construction where improved quality control can be used to ensure very accurate and high-quality engineered products.
- In areas of high-humidity or aggressive environments such as swimming pools.
- Architecturally expressive details and/or unfinished structural framing.

See Chapter 19: *Timber*, for more information.

16.4.4 Masonry

Structural masonry exhibits the following material in-service characteristics, and structural design and calculation of movements should account for these issues:

- Applied load deformation is elastic and in some cases time-dependent; brittleness precludes design in the plastic range.
- Response to environmental changes and applied loads is broadly isotropic, although unidirectional cracking is common in incorrectly designed and constructed structures.

- Response to low temperatures (i.e. non-fire) loads is elastic.
- Shrinkage, mortar strength gain and cracking are all affected by environmental humidity.
- Masonry properties are heavily material dependent; concrete, clay and calcium silicate units have significantly different in-service behaviour and differing detailing requirements.
- Element degradation response is expansive in case of freeze–thaw cracking.
- Creep is dependent on mortar and unit type; clay bricks with lime mortars are inherently more flexible than cement mortars and concrete units.

Structural masonry has three types of dimensional tolerance:

- Manufacturing tolerances: constituent materials and manufacturing are naturally variable.
- Environmental tolerances: dependent on material type and exposure.
- Erection tolerances: dependent on site construction quality control.

European and American codes of practice and material standards are well developed with respect to manufacturing, environmental and placement tolerances. In most cases the inherent improbability of all unfavourable extreme deviations occurring together is small since masonry is straightforward to adjust in the field. See Section 16.10 for further references.

Carefully specified movement joints, ties, joint reinforcement and head-restraints that facilitate controlled expansion and contraction are required to avoid uni-directional cracking in the plain of a wall. In seismic areas masonry infill of structural frames should have compressible joints to avoid the masonry becoming part of the lateral load resisting system.

Tighter specification of movements and tolerances may be required in the following cases:

- Multi-storey lift shaft constructions: wall verticality and horizontal deflection under wind load need to be carefully controlled in line with lift manufacturer specifications.
- Architecturally exposed or unfinished masonry: high-quality panels and joints.

Moreover masonry working practices and material types vary considerably around the world, making local knowledge vital in terms of:

- permissible and likely deviations;
- jointing and pointing practices;
- storage, preparation and use of materials on site;
- masonry protection during execution.

In the case of pre-fabricated panels and calcium silicate large blocks off-site pre-fabrication can be readily used to achieve tighter tolerances but care is required as they may be too onerous for the actual site conditions. The means for local adjustment should always be provided.

See Chapter 20: *Masonry*, for more information.

16.4.5 Aluminium

Structural aluminium exhibits the following material in-service characteristics, and structural design and calculation of movements should account for these issues:

- Applied load deformation is linear elastic up to the yield stress and plastic thereafter.
- In welded structures, aluminium alloy mechanical properties vary significantly between the parent metal, weld metal and the heat affected zone (HAZ).
- Characteristic design strengths can be increased in proportion to increased testing.
- Response to environmental changes and applied loads is isotropic.
- Response to low temperatures (i.e. non-fire) loads is linear elastic; in particular it is suitable for cryogenic applications as unlike steel it is not prone to brittle fracture at low temperature and its mechanical properties steadily improve with decreasing temperature.
- Non-corrosive response to environmental humidity changes is negligible.
- Element corrosion response creates a thin inert protective coating of aluminium oxide, although serious electrolytic corrosion may occur at unprotected joints with other metals.
- Creep rate is low and implicitly included in codified stress–strain–temperature formulae.
- High creep rates in fires are dependent on rate of heating, temperature and applied member stress.
- Fatigue failure is a key design constraint under cyclical loading.

Structural aluminium has three types of dimensional tolerance:

- Manufacturing tolerances: extrusion/plate thickness, flatness and section dimensions.
- Fabrication tolerances: dependent on workshop quality control.
- Erection tolerances: dependent on site construction quality control.

European and American codes of practice and material standards are well developed with respect to manufacturing, fabrication and erection tolerances as referenced below. The fact that aluminium is relatively easy to mould means that project-specific extrusions are often fabricated, unlike structural steel where standard sizes and shapes are very common. In most cases, the inherent improbability of all unfavourable extreme deviations occurring together is small, and simple means of on-site adjustment can be incorporated to avoid the cumulative accumulation of deviations. These include top-hung facade panels, packing pieces, slotted holes and threaded rods. Fit tolerances tend to be governed by conditions in the field, so accurate pre-erection surveys of support structures should be undertaken prior to erection. See Section 16.10 for further references.

In most respects, the design processes of aluminium and steel structures are similar, which in particular explains the sharing of Eurocode Execution Codes. However, there are noteworthy differences in their physical and mechanical properties which must be accounted for in the design process.

- The heat input in welded aluminium profiles eliminates some of favourable consequences from heat treatment or strain hardening. This decreases the local elastic limit resulting in strength redistribution along the cross-section profile.
- The coefficient of linear expansion is higher than that of steel.
- The low density of aluminium and its high strength to weight ratio are the main drivers for its use. These are broadly favourable but they do present disadvantages; in cyclical loading conditions the ratio of live/dead load is low as compared to steel making fatigue design critical; its low density also makes an aluminium structure prone to vibrations so dynamic behaviour of the structure must be considered.
- The aluminium Young's modulus is about one-third that of steel; for equivalent steel and aluminium sections its deflections are therefore proportionally higher; combined with its low density, this also lowers the fatigue strength of aluminium to about half that of steel.

Given these constraints, and the fact that aluminium designs often use slender, thin walled sections, torsional buckling, shear deformation and shear stability are key design constraints.

Tighter specification of movements and tolerances may be required in the following cases:

- Facade structures to high-rise towers with large deflections.
- Dynamically loaded facade or canopy structures with fatigue-sensitive connections via quality control of welding and connections.
- Highly architectural features such as glass stairs, floors and eye-level facades.

16.4.6 Glass

Structural glass exhibits the following material in-service characteristics, and structural design and calculation of movements should account for these issues:

- Applied load deformation is elastic and time-dependent; brittleness precludes design in the plastic range.
- Response to environmental changes and applied loads is isotropic.
- Response to low temperatures (i.e. non-fire) loads is linear elastic.
- Laminated panel interlayer strength can degrade in high humidity conditions.
- Panel strength is time dependent and decreases with load duration.
- High exposure to solar radiation, warm-air stratification beneath panels and/or other types of sustained loading can cause interlayer creep.

Structural glass has three types of dimensional tolerance:

- Manufacturing tolerances: dependent on factory quality control.
- Environmental tolerances: dependent on exposure and use of material.
- Erection tolerances: dependent on site construction quality control.

European and American codes of practice and material standards are well developed with respect to manufacturing, environmental and placement tolerances as referenced below.

In most cases structural glass is fabricated in a controlled factory environment, allowing accurate tolerances to be achieved. Fit tolerances tend to be governed by conditions in the field, so accurate pre-erection surveys of support structures should be undertaken prior to erection. See Section 16.10 for further references.

The brittle behaviour of glass and its potential for accidental cracking means that potential panel failure and robustness criteria must be assessed during design such that:

- glass panels must be replaceable;
- panel choice and its integration into a structure depend on its potential failure consequences; progressive collapse must be avoided by providing residual structural capacity after failure to reduce post-cracking hazards.

This usually requires one of the laminates to be heat-strengthened glass, designed to support loads in a temporary conditions.

Tighter specification of movements and tolerances may be required in the following cases:

- high-rise towers and dynamically loaded facade or canopy structures;
- highly architectural features such as glass stairs or floors.

See Chapter 21: *Structural glass*, for more information on glass.

16.4.7 Membrane tensile structures

Membranes exhibit the following material in-service characteristics, and structural design and calculation of movements should account for these issues:

- Applied load deformation is elastic and plastic above and below yield stress.
- Response to environmental changes and applied loads is anisotropic.

- Response to low temperatures (i.e. non-fire) loads is linear elastic.
- Shrinkage and strength are all affected by environmental humidity.
- UV light response, fire and weather resistance are material and location specific.
- Creep is dependent on direction of applied load in relation to fibres.

Membrane structures have three types of dimensional tolerance:

- Source tolerance: constituent materials naturally variable, with structural performance dependent on type of textile or membrane used.
- Manufacturing tolerances: dependent on factory quality control and pattern generation.
- Environmental tolerances: dependent on exposure and use of material.
- Erection tolerances: dependent on site construction quality control.

European and American codes of practice, material standards and working practice guides are still developing, with American codified practice more developed and European codes still under development. Best-practice guidance and working practices are most often based on design consultant and manufacturer-specific data and systems. See Section 16.10 for further references.

Tensile structures are those with no compression often designed with unique and aesthetically appealing arrangements, with compression resistance provided externally by steel columns or supports. Some are built up from a woven textile base whilst others are pure plastics. Typical membranes include polyester or titanium dioxide coated polyvinyl chloride (PVC), glass fibre coated polytetrafluoroethylene (PTFE), high density poly ethylene (HPDE) meshes and ethyl tetrafluoroethylene (ETFE). Working practices and facilities management practices vary considerably around the world making local knowledge vital in the specification of materials in terms of:

- likely maintenance regimes or lack thereof;
- contractor capability in membrane manufacture and cable-jacking.

Construction is often bespoke to meet specific architectural, ultraviolet light protection, fire and weather resistance requirements, so design must account for movement and tolerances impacts across the major system components:

- Tensile membrane fabric: inherent material deviation, pattern generation and seam joints.
- Support structure: rigid frames, suspension/cable net systems or air-support systems.
- Connection system: clamps and plates, bale rings and caps, ridge/valley, boundary/link and tie-back/tie-down cables.

Construction-stage and in-service deflections are high requiring complex form-finding and nonlinear analysis to come up with an optimal cutting pattern that is not susceptible to wind shear and galloping instabilities. All structural details and interfaces with fixed structure must accommodate these deflections, rotations and movements. Many tension structure failures have occurred when the fabric movement has allowed it to get snared by other parts of the structure, creating a membrane tear failure. Thus, deflection must be free to occur.

User perception is often used to impose a practical limit on large deflections if they are close and visible, although occupants in a tent-like enclosure can tolerate larger movements as these are somewhat expected. Large membrane roof deflections can also actually change the internal pressure in an enclosed space which can be a possible problem.

16.4.8 Structural composites

Structural composites such as fibre-reinforced polymer (FRP), glass-reinforced plastic (GRP) and other forms of carbon fibre composites exhibit the following material in-service characteristics which should be accounted for in structural design and the calculation of movements:

- Applied load deformation is elastic and heavily influenced by grain/fibre direction; brittleness precludes design in the plastic range.
- Response to environmental changes and applied loads is anisotropic.
- In-service temperature and moisture content changes lead to thermal or hygroscopic residual stresses inside the laminate which must be accounted for in design.
- Element strength is time dependent and decreases with load duration.
- Matrix strength can degrade in high humidity or high solar radiation conditions.
- Response to low temperatures (i.e. non-fire) loads is linear elastic.
- Post-fabrication shrinkage is very low.
- Fatigue failure is possible under cyclical loading.
- Creep is dependent on material type and direction of applied load in relation to fibres, and is generally considered to be low.

Structural composite structures have four types of dimensional tolerance:

- Source tolerance: constituent materials naturally variable, with structural performance dependent on type of composite used.
- Manufacturing tolerances: dependent on factory quality control and pattern/mould control.
- Environmental tolerances: dependent on exposure and use of material.
- Erection tolerances: dependent on site construction quality control.

Composite structural materials are most often used to make complex thin shapes where weight carries a premium, most often in the aerospace and competition racing industries. In the construction industry they are most prominent in complex facades, dome roofs and feature structures such as the Dokaae Tower in Mecca, where the upper 370 m of cladding including clock faces and hands was constructed using structural composites (see **Figures 16.1** and **16.2**). Their design is much more complex than that seen with standard construction materials, and continuous innovation in the field means that materials and design methods are in a constant state of evolution. Best-practice guidance and working practices are most often based on design consultant, academic research and manufacturer-specific data and systems. Therefore, design is best undertaken by specialists.

All structural composites are fabricated in a controlled factory environment allowing accurate tolerances to be achieved. Fit tolerances tend to be governed by conditions in the field, so accurate pre-erection surveys of support structures should be undertaken prior to erection. Support structures must also be designed to accommodate low out-of-plane stiffness and potentially brittle behaviour of structural composites.

16.5 Foundation movement

It is important that structural engineers have a thorough grasp of any issues that could affect their building or structural

Figure 16.1 Dokaae Clock Tower, Mecca, KSA: clock hand being lifted into position (Courtesy of Premier Composite Technologies)

Figure 16.2 Dokaae Clock Tower, Mecca, KSA: carbon fibre clock faces under construction (Courtesy of Premier Composite Technologies)

foundation design. The foundations for any structure must be designed to have adequate load capacity with acceptable levels of settlement, which means that foundation design is often an iterative exercise undertaken in unison by a geotechnical and a structural engineer.

The key issues that need to be understood by these two parties are as follows:

- Soil profile and design characteristics as interpreted from a site-specific soil investigation report.
- Water table profile, to include an understanding of long- and short-term level changes if relevant.
- Local activities such as mining, oil, gas or water extraction that may affect foundation design.
- Building or infrastructure adjacencies that may affect foundation and building design.

Once this understanding is in place, a foundation system can be chosen that balances the risks of damage against project capital cost. An incorrectly installed foundation system can be costly to remedy, so the project client, local authorities and other project team stakeholders should be clear on how the building is likely to behave once constructed.

The term 'behave' is used because *all* buildings move, yet it can be difficult to fully quantify the level of settlement, its timescale, the risks and life-cycle costs that this could provoke. Moreover, it is also difficult to fully quantify the term 'failure' as the term is highly subjective; minor structural cracking and load redistribution of a structural frame might be perfectly acceptable to the structural engineer, but if the facade rotates and cracks the architect and client are less likely to share the same opinion.

For this reason it is imperative that the structural engineer, in collaboration with a geotechnical engineer, fully considers how a foundation is likely to react to load and its environment over time. Once this is understood, this information can be communicated to the remainder of the design and client team so that the building can be cost-effectively designed and constructed to accommodate this movement within an agreed set of risk and tolerance boundaries.

16.5.1 The causes of foundation movement

Since all foundations settle, an engineering assessment of the site geotechnical condition and its potential risks should be undertaken. There are four main risk categories:

- inherent geological risks that can be exacerbated by construction activity;
- water-induced risks that can affect soil behaviour and/or building loading;
- industrial activity or interferences within the local bearing strata;
- the construction process and its interaction with the ground and its surrounding buildings.

These risk categories can be broken down into the following broad reasons for foundation-related movement.

16.5.1.1 Bearing capacity

In common with other construction materials, soils deform under loading. Soil bearing capacity is the measure of its capacity to support loads, with ultimate bearing capacity used as the theoretical maximum pressure that can be supported without failure; either general shear failure, local shear failure or punching shear failure. Weaker soils tend to settle more under loading, often without shear failure. In such cases, a working stress allowable bearing capacity is determined by the geotechnical and structural engineer to ascertain a maximum allowable settlement. This value is often governed by external constraints, such as tolerable structural frame movements or facade tolerances.

Soils also deform at differing rates; in general terms granular soils experience most of their deflection during construction, whilst cohesive soils can be prone to slow acting creep effects.

16.5.1.2 Slow soil creep in expansive soil

Swelling rates can vary across a complete foundation due to seasonal or long-term changes in water levels – often prolonged periods of wet or dry weather – or due to localised vegetation removing moisture. It is an unavoidable issue when constructing on clay soils, so understanding how changes in moisture content could affect the soil is of vital importance as foundation distortion can result in structural movements that must either be mitigated or accommodated.

16.5.1.3 Inadequate site drainage underneath or around the structure, or behind its retaining walls

Uncontrolled water flow from leaking drains can have two principal effects. In cohesive or expansive soil such as clay it can change the moisture equilibrium within the soil, which could cause heave. In a granular soil, the water can wash out fine particles leading to gradual and sometimes highly localised consolidation of the remaining soil.

16.5.1.4 Groundwater movement and flooding

Groundwater levels are often seasonal, those near to the coast are tidal, lunar and seasonal, and in some areas they could trend upwards or downwards depending on local groundwater extraction or charging trends. Additionally, flooding can be a risk in many low-lying areas. These issues must be fully quantified by the structural engineer so that the design can be prepared appropriately.

16.5.1.5 Improper or inadequate site preparation, grading or compaction

This is a particular problem where shallow foundations are supported in the uppermost bearing strata. Insufficient or inadequate site preparation can leave a building or structure susceptible to uncontrolled settlement as the supporting soil consolidates.

16.5.1.6 Inadequate pile depth

Older buildings supported by wood or early steel pile foundations may encounter problems when these do not extend below the zone of expansive soil that is affected by the climate or changes in water level. Their shallowness may not provide sufficient restraint to foundation settlement or heave.

16.5.1.7 Degradation of foundations

Old timber foundations, and in some cases RC or steel piles, may be prone to corrosion. All types of foundation are very resilient if kept below the groundwater level as oxygen to feed the degradation process is generally lacking despite ample quantities of water. Confirmation of wetting and drying cycles is therefore required to predict potential long-term degradation and unforeseen building movements.

Degradation can also be initiated by high levels of sulphates, chlorides and other deleterious ions in groundwater, which can attack reinforced concrete and masonry structures (see **Figure 16.3**). Analysis of site groundwater contamination

Figure 16.3 Groundwater salt attack on a masonry wall; the stronger mortar is left *in situ* whilst the blocks have slowly degraded, Ras Al Khaimah, UAE (Courtesy of Buro Happold)

should be undertaken to ensure that adequate protective measures can be implemented.

16.5.1.8 Subsidence due to mining

This is relatively predictable in magnitude, location and extent unless an old mine working suddenly collapses. The effects tend to be local to the area immediately above the mine.

16.5.1.9 Gas, oil exploration or water extraction

Extraction of any liquid from a porous rock medium reduces the internal pressure within it, which in turn reduces the support that was given to the soil layers above. Once the internal liquid pressure drops the soil pressure increases and this leads to subsidence at the ground level. The size and extent of these subterranean wells tend to be large, meaning that subsidence at ground level is consistent and predictable across a wide area.

This effect can be replicated in areas with peat soils when water extraction lowers the local water table. The de-saturation of the peat allows oxygen to mix with the peat causing it to slowly decompose and reduce in volume.

16.5.1.10 Landslide on mountainous or hilly terrain

This is a geological ground movement phenomenon of which there are a number of types: rockfalls, landslides, deep-seated failure of slopes and shallow debris flows. They can occur in offshore, coastal and onshore areas, and this ground destabilisation can have many differing causes: changes in groundwater pressure; loss or absence of forest or vegetative structures; toe erosion of a slope by rivers or the ocean; slope weakening by snow/glacier melt or heavy rains; earthquakes or human-induced destabilisation by blasting or earthworks.

16.5.1.11 Soil dissolution

A frequent issue in karst terrains is subsidence due to the dissolution of limestone by groundwater flow. This creates cavities and large caves, a process which is often accelerated by the mixing of water and atmospheric carbon dioxide to form a weak acid called carbonic acid. If the roof of these voids becomes too weak, or its load patterns change due to new surface-level construction, it can collapse causing subsidence or sinkholes at the surface. Other lower-risk forms of soil dissolution exist, including that of gypsum along its boundary layers when there is water flow. This makes it important that site conditions be reviewed by a geotechnical engineer (see **Figure 16.4**).

16.5.1.12 Foundation scour

This can occur when flowing water removes the supporting soil from around a footing, particularly around bridge piers,

Figure 16.4 Building movement due to limestone cavity collapse, Abu Dhabi, UAE (Courtesy of Buro Happold)

wind turbine mono-piles and other marine structures. As water flow accelerates to get around an obstruction, the increased turbulence can scour the surrounding soils.

16.5.1.13 Frost heave

This occurs when water in the ground freezes to form expansive ice lenses that can damage adjacent foundations and structures. Ice initially grows in the direction of heat loss vertically towards the surface, starting at the frozen boundary within the soil. Once the freezing frost has sufficiently penetrated the soil horizontal growth can begin. A water supply is required to feed ice crystal growth, and whenever the growing ice is sufficiently restrained by overlying soil lens-shaped areas of ice propagate. Soils that are vulnerable to these effects are deemed to be *frost susceptible*, and due consideration of these effects must given to adjacent buildings and their foundations.

16.5.1.14 Permafrost melting

This must be considered in the event that a building is built on such ground, with a particular requirement to limit heat loss that could melt its immediate foundation supports.

16.5.1.15 Soil liquefaction

This is observed during a seismic event, whereby saturated soil substantially loses strength and stiffness in response to the applied stress from shaking causing it to behave like a liquid. It is a phenomenon most often observed in saturated, loose, sandy soils and its effects have long been understood and accounted for within building design codes. A site-specific engineering understanding of this risk is required so that the structure can be designed to resist the higher earthquake peak ground accelerations that this soil condition produces. It will not manifest itself during normal in-service use, but in the event of a strong earthquake the building will experience magnified building movements if the soil liquefies.

16.5.2 Building and infrastructure adjacencies

Projects that require deep excavations or foundations in urban areas must often take into account the response of adjacent buildings, tunnels and utilities to excavation-related ground movements. The structural engineer, in conjunction with a geotechnical engineer, must evaluate these interdependent responses using a combination of theoretical and empirical methods to set limiting criteria to safeguard all buildings and infrastructure against unacceptable damage. Movements cannot be avoided, so this estimation of building responses and the subsequent severity of excavation-related building damage (i.e. cracking or visual movement) is critical to establishing rational limiting criteria for excavation support system and building designs.

From a structural engineering perspective it should be noted that flexible old buildings are often better able to cope with movement than their more modern rigid counterparts. The prevalence in older buildings of soft lime mortars, massive walls, timber-frames, arches and vaulted construction make them inherently more flexible. Modern structures with slender masonry walls set in hard cement mortars will show every crack in their brittle plasters.

16.5.3 Settlement as a design and construction goal

A thorough review of foundation and construction risks will sometimes reveal that controlled construction settlement and structural movement are a desirable design and construction goal. This is particularly notable when designing raft or bearing-type foundation structures, as these often present a cost-effective means of supporting loads and dealing with potential soil movements.

In extreme cases, most often in high-rise buildings such as the Burj Khalifa in Dubai and The Landmark tower in Abu Dhabi, this soil–structure interaction can be closely modelled to create a piled raft foundation where movement is both necessary and highly desirable. The raft foundation mobilises the ground pressure under the raft in unison with strategically placed piles that redistribute loads to achieve a more uniform raft settlement at working load. This improves the foundation settlement profile and distributes load more efficiently to maximise local ground-bearing potential.

16.5.4 Risk evaluation of potential foundation movements

A thorough review of the potential causes and limitations of construction movement must be focused on:

- soil restrictions in terms of geological risks, bearing capacity and settlement;
- water flow regime and its potential impacts;
- adjacencies and other external short and long-term risks.

Once identified, they should be assessed using the basic principles of risk management:

1. Identification of risks (i.e. expansive soil potential).
2. Assess the vulnerability of the project components to this risk (i.e. facade movement).
3. Determination of risk impact and a risk reduction hierarchy (i.e. either install tension piles and design structure to resist heave effects, or design structure to allow movement and cladding connections to allow rotation).
4. Formally communicate the risks and options to the client and wider project team.
5. Review the risks and impacts with team to agree mitigation measures (i.e. rotation of cladding has a high probability of occurrence and would appear unsightly to the client and end users making it unacceptable; this requires structure to be designed and constructed to resist soil heave effects despite its higher capital cost).

A risk and impact-based approach that involves the client and other design members ensures a more open and holistic approach to decision-making, allowing all parties to better understand and manage potential impacts.

Whilst settlement cannot usually be avoided, the structural engineer should have options available that can minimise deleterious movement effects to an absolute minimum. In practice foundation systems can be deliberately designed to have very low working stresses using a large raft structure or large RC piles. Alternatively the supporting soil can be modified to improve its bearing capacity and settlement characteristics using ground improvement or injection techniques such as dynamic compaction or jet-grouting.

Since uniform settlement of a building structure is rarely a predominant issue the benefits of adopting the low working stress techniques noted above rarely justify their capital costs. In terms of settlement and understanding building movement the engineer will more likely have to focus attention on the following:

- High-risk low probability events such as earthquakes.
- High-probability high-risk events such as changing water tables or expansive soils.
- Differential settlement between one portion of the structure and another, especially if this impacts on architectural finishes, building services, facades and other trades.
- Settlement impacts on adjacent buildings or infrastructure.

16.5.4.1 Case study: The Landmark, Abu Dhabi, UAE

Key design facts

- 329 m high mixed-use tower with five storey basement structure (see **Figure 16.5** for an image of the tower under construction);
- RC superstructure with steelwork top of tower and canopy structures;
- Structure designed and detailed to resist environmental seismic and wind loads.
- Differential shortening of columns and walls checked as part of original design, with emphasis on review of RC structural elastic, plastic and creep settlements that are affected by high local humidity and temperatures.
- Transfer structures, slabs and all secondary elements designed to accommodate differential movements at their supports.
- Foundation designed as a pile-assisted raft with controlled settlement due to difficult ground conditions; elastic and plastic settlement risks identified, with low risk of gypsum soil dissolution.
- Steelwork structures designed for fatigue loading and differential movements.
- In-service lateral load deflection and acceleration a key design criteria.
- Tolerances specified according to material within the relevant specification.
- Detailed movement and tolerances report issued as part of tender package to clarify expected in-service deflections for coordination on non-structural interfaces such as facades, building services and lifts.

Figure 16.5 The Landmark under construction in Abu Dhabi, UAE (Courtesy of Malcolm White/Buro Happold)

Key construction facts

- Contractor temporary works loads and deflections from cranes, pumps and hoists reviewed as part of construction design checks.
- Contractor-prepared axial shortening study undertaken to predict construction stage settlement in line with planned construction sequence.
- Detailed system of construction surveying and monitoring systems used to verify movement and tolerances during construction in line with contractual specifications.
- Principal setting-out undertaken in the early morning as diurnal heating and cooling of exposed building structure caused it to rotate due to differential heating effect of sun warming it on one side.
- Installation of non-structural items such as facade, lifts and building services made use of time-dependent settlement analysis to minimise deleterious impacts.

Principal means of monitoring construction-stage movements and tolerances

Contractor systems used as specified in the contract specifications:

- Cast *in situ* vibrating wire strain gauges within superstructure columns and walls at every five floors.

- Inclinometer system on five floor levels.
- Rod-type borehole extensometers below the tower foundation raft.
- Earth pressure load cells below the tower foundation raft.
- Cast *in situ* vibrating wire strain gauges within the foundation piles.
- Groundwater piezometers.
- Anemometers to monitor wind loads.
- Target surveying points placed on raft and every five floors thereafter to monitor in-service movements on a bi-weekly basis.
- Sophisticated GPS setting-out system used in conjunction with traditional theodolite, laser plumb and weighted measurement tapes to guarantee setting-out.
- See **Figures 16.6** and **16.7** for position references.

Principal means of monitoring post-construction movements and tolerances

Systems installed in the contract specifications for construction stage supplemented by:

- Accelerometers to measure dynamic lateral/torsion building response.
- Building monitoring system to process results.

16.6 Strategies for dealing with movement and tolerance effects

16.6.1 Prevention of movement

The prevention of all structural movements is theoretically possible, but in most cases there is little cost or technical benefit in doing so. A better approach is to extend the foundation-specific risk analysis identified in Section 16.5.4 across the whole the building structure to identify all potential movement risks. These can then be brought to the attention of the team so that cost-effective mitigations and allowances can be made within design and construction to alleviate the potential problems.

16.6.2 Minimisation of effects

Mitigation strategies require an understanding of the potential problems, their impacts and remedies. As noted below, this can be used to determine cost-effective solutions:

- Standard material and element deviation: set tolerances to suit known deviation ranges.
- Moisture control: use insulation, coatings and a preventative maintenance regime to control moisture ingress, and design/detail the structure for the most likely range of moisture scenarios.
- Temperature control: use insulation to control temperature differentials during construction and in-service, and design/detail the structure for the most likely range of temperature scenarios.
- Shrinkage control: plan construction of shrinkage-prone structures to allow movement without distress; for example, an allowance for construction stage pour strips in RC construction to allow early-age movements; or in deflection sensitive areas non-structural cladding and building services can be fixed after settlement has occurred, or be provided with connection joints that allow movement.
- Loading: reduce element loads by refining load criteria.
- Response to loading: design/detail structure and plan construction to account for elastic, plastic and creep responses.
- Structural materials, sizing and structural layout: choose materials that are less movement sensitive; amend sizes to reduce movement amending the structural layout to reduce or redistribute loads; pre-set vertical element lengths to account for construction and in-service movement, or pre-camber horizontal elements to allow for dead load deflections.
- Amend structural size/shape: some forms of movement can be cost-effectively isolated by providing permanent movement joints to create effectively separate building structures.
- Bearings, dampers and restraints: some movements and rotations are better accommodated using elastomeric, seismic isolation or slide bearings, some of which include sophisticated fluid isolation damping and seismic lock-up devices; in high-rise towers tuned mass and active/passive fluid dampers can be used to control lateral deflection.
- Design and detailing for movement and tolerances: design and detail the structure to account for the balance of movement and required tolerances.

In most cases this early appreciation can be used to design, detail and construct out the problems.

16.6.3 Pre-fabrication

Pre-fabrication of building components in an off-site manufacturing centre for on-site assembly can assist in controlling movement and tolerances effects. This has two principal benefits: improved quality control and the avoidance of any movement that results from the construction process. The improvement in size tolerance is material independent, as structures can be assembled as accurately as required. In the case of concrete or timber structures, pre-construction allows some time-dependent properties such as drying shrinkage and creep to take place independently prior to assembly.

16.6.4 Building information modelling and virtual proto-typing

Building information modelling (BIM) is the process of generating and managing building data for use during its life-cycle. In practice, this database model can be linked to a series virtual design and simulation tools that effectively create a virtual prototype of the site, buildings and all necessary adjacencies. This allows any aspect of the design and construction to be simulated and assessed before it is built challenging the intended design more completely and much earlier. Current three-dimensional (3D) design and analysis models are evolving to become 4D with construction programme and time added in, 5D with construction quantities and costs, and 6D with in-service performance monitoring and assessment. As this technology further evolves, it will give contractors the ability to virtually

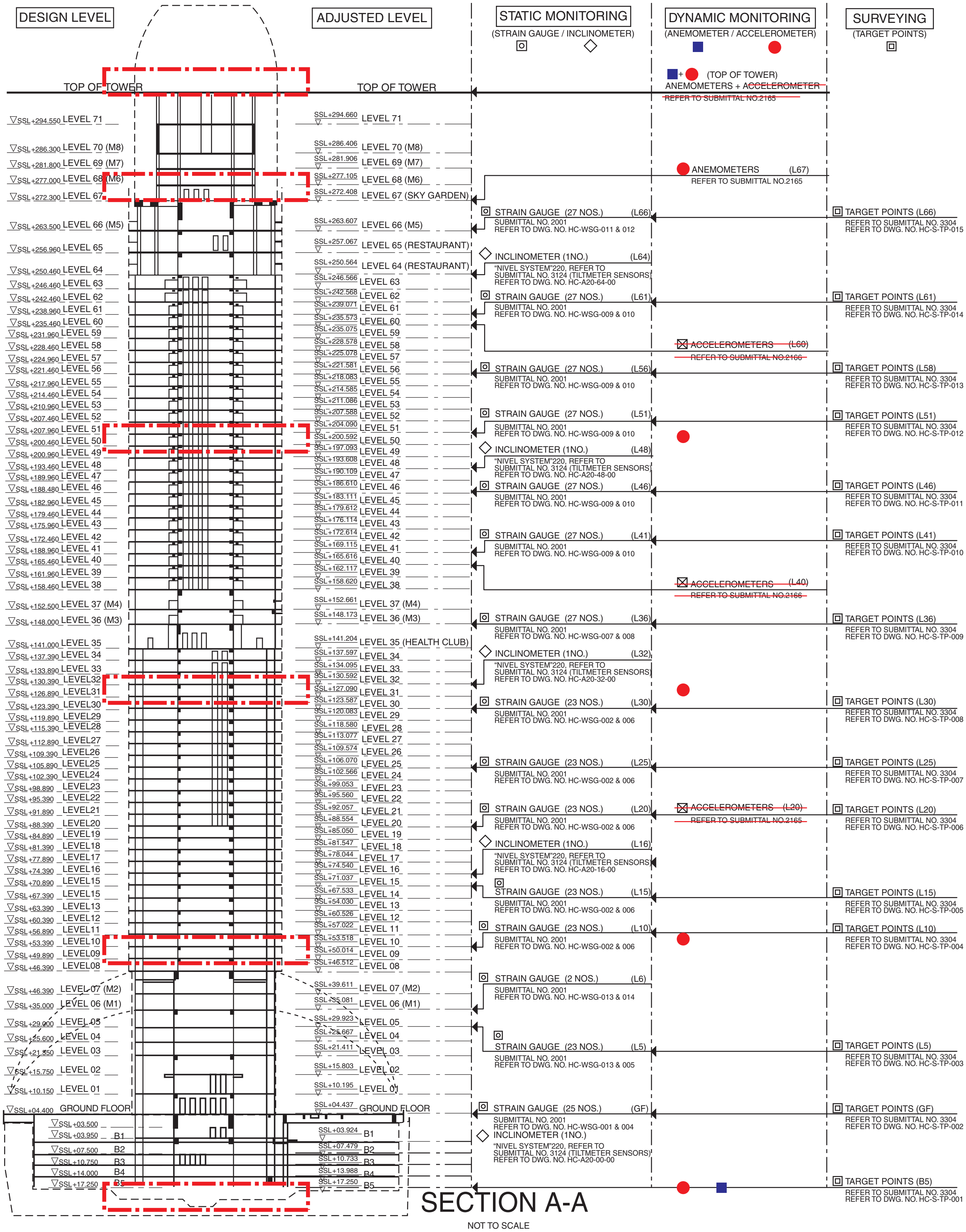

Figure 16.6 Cross-section through the tower showing positions of all surveying and monitoring stations (Courtesy of Buro Happold)

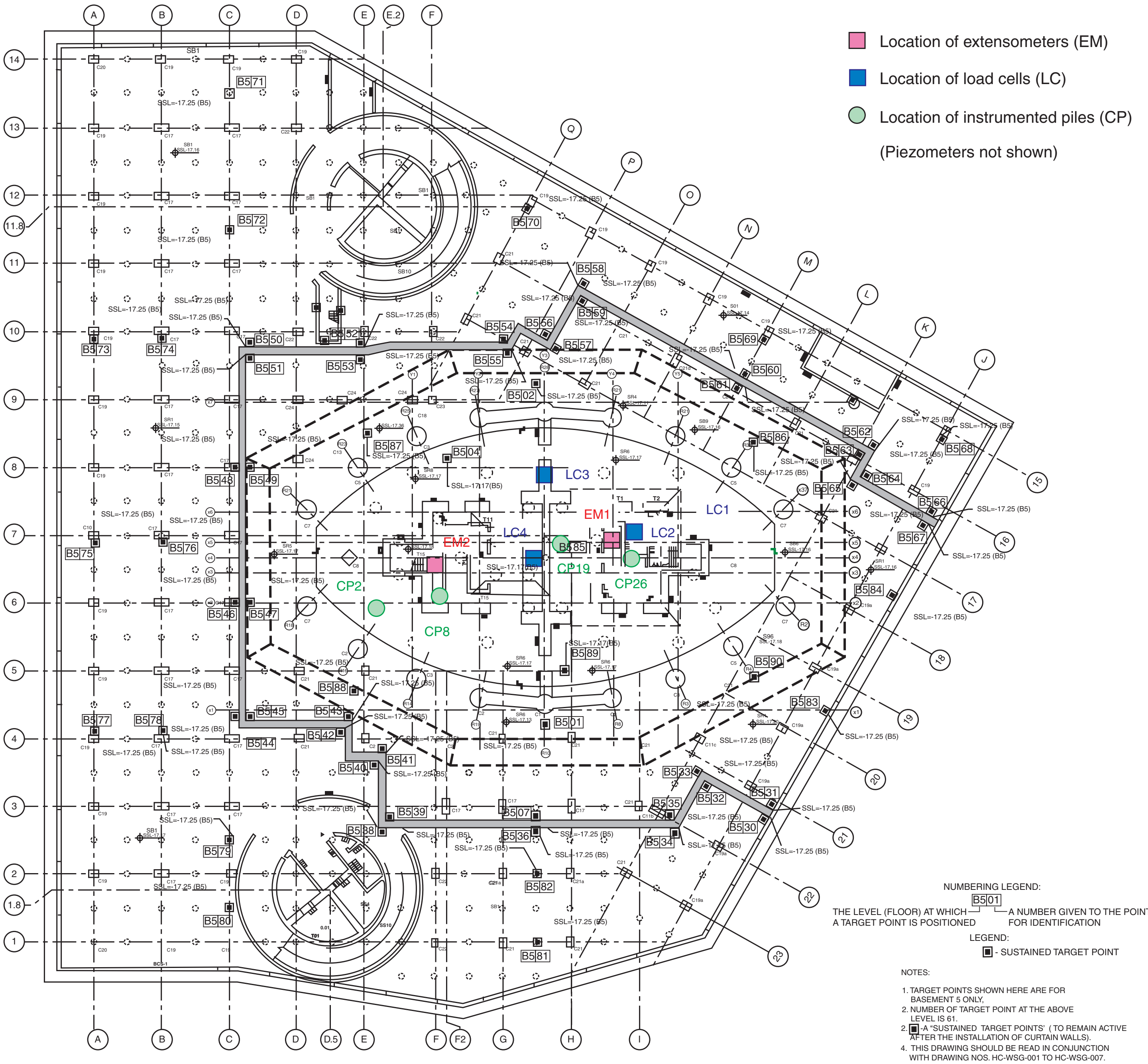

Figure 16.7 Plan view of raft structure showing positions of all surveying and monitoring stations (Courtesy of Buro Happold)

construct an entire building and clients the opportunity to manage their assets more efficiently, with the potential to input this best-practice learning back into the design process.

From a movement and tolerances perspective, BIM allows designers and constructors to virtually collaborate to reduce design conflicts, produce more efficient and optimised designs, and fast-track schedules with the goal of increasing the sustainability of construction and operation.

16.6.5 Standard methods of design for movement

Codified structural design methods are clear in their treatment of structural movements, and it is not uncommon to review the secondary shear and moment effects induced in members due to structural displacement under vertical and lateral loads. These can be element- or location-specific, or may require analysis of the whole structure to assess side-sway collapse. This can occur when the effective storey shear due to inertial forces and p-delta effects – an additional column bending stress caused by eccentric vertical load application – exceeds the storey shear resistance. In specialised cases, this may require nonlinear analysis of each load combination so that all potential collapse mechanisms are identified and resolved.

16.7 Methods of measurement and control

16.7.1 Units of measurement

The International System of Units (abbreviated SI from the French: système international d'unités) is the modern form of the metric system, based on the metre–kilogram–second system. It is commonly used throughout the world. The only other alternative form of measurement in common use is the imperial system (sometimes known as British Imperial) which was extensively used throughout the British Empire. The USA is the only industrialized nation that does not officially use the metric system, although some commercial property transactions still use the system in the United Kingdom, Australia and Hong Kong.

Care is required when converting between imperial and metric units, with large inaccuracies possible if incorrect conversion calculations are used. It is advised that projects be started and finished using the required unit system to avoid the need for conversion and likelihood of potential problems.

Otherwise it should be noted that different countries present drawings and metric units in different ways; some present drawings in millimetres and others in centimetres, and forces in tonnes (kilograms) instead of Newtons.

16.7.2 Setting-out and inaccuracies

Setting-out of existing and new structures requires linear and angular measurement to establish vertical and horizontal control. The accuracy achieved depends on the measuring equipment used, as well as the knowledge, ability and diligence of the operator.

The right equipment must be used for the task. Small construction projects and portions thereof can be accurately set-out using a theodolite, level and the 3:4:5 triangle rule, whilst more complex and sizeable projects will likely require and benefit from more advanced techniques that use global positioning system (GPS) satellite technology. In-field limitations and special setting-out requirements should be recognised when preparing the contract documentation so that tenders are more realistically prepared and priced. In addition, the specification of specialist setting-out techniques should include a description of how this should be planned, monitored and potentially corrected to avoid potential disputes in the event of construction non-conformance. See Section 16.10 for further references.

16.8 Dispute resolution

Construction disputes are generally caused by one of the following:

- Expectation clashes, most often entrenched during the tender process, and most prominent when one party has been overly opportunistic in contract negotiations, with the other being excessively aggressive, or perhaps optimistic, in pricing.
- Unrealistic allocation of risk.
- Contractual ambiguities.
- Failures in communication and contract administration, often leading to arguments on the necessity for interim extensions of time, or monetary release/relief.
- Failure to recognise and close out problems properly as they arise, especially scope ambiguity or contract modifications.

In the context of construction movement and tolerances, dispute resolution is best dealt with in the original contract documentation by ensuring that items are clear, reasonable and coordinated. Litigation and its associated alternative forms of dispute resolution should be avoided at all costs, primarily through preventative actions and a flexible approach to dispute resolution.

Preventative actions can be grouped as follows:

- Contract drawings, specifications and reports should be clear on all anticipated movement and tolerances issues to avoid disputes between the client and the contractor, subcontractors and design professionals on interpretation of the contractual documents, particularly if the documents contain contradictions and ambiguities. Typically, there is an implied warranty on the part of the owner that the documentation is correct, coordinated and buildable, which is often counterbalanced by clauses in the contract documentation that attempt to shift the coordination and buildability risks and responsibilities onto the contractor. Any resulting confrontation therefore revolves around the implied warranty and enforcement of the exculpatory clauses in the documentation.
- In all cases the design documentation should be clear on areas requiring special accuracy with respect to details, joints and interfaces, with consideration and specification of construction trials and mock-ups of particular areas of concern, with a clear explanation and images of how conformance will be measured, monitored and corrected during the works.
- Scopes of work should be clear and complete, particularly with respect to design consultant and contractor responsibilities, and

consequential scopes of work between the contractor and its subcontractor. Scope gaps often create disputes.

- Uncoordinated contract design documentation, or a failure by the contractor to properly coordinate its subcontractors, can lead to disputes that revolve around consequential delays and third party coordination impacts. An emphasis on thorough coordination and active use of electronic clash detection software within the BIM environment can mitigate these impacts.

Pre-construction meetings can be used to great effect to focus attention on these issues, but when disputes inevitably flourish a flexible and positive approach to dispute resolution is needed:

- A request for information (RFI) process is normally set up to handle interpretational problems related to the contractual documentation, plans and specifications. If the parties to the process work in the right spirit it can be used to flexibly resolve disputes and avoid conflict. However, the RFI process can be misused by contractors to create additional and unnecessary time and cost claims, to mask inadequate contract planning or to corner the project team by delaying the issue of RFIs until they are time-critical. This can be exacerbated by client and design professional failures to acknowledge faults and anomalies, and overlaps between project design and construction means/methods that have not been clearly defined in the project scope of works.
- Unnecessarily strict interpretations or changes to accommodate movement and tolerances issues should always be assessed against their time and cost impact, especially if they could have a cumulative impact on other activities. In such cases, relaxation of criteria or alternative means of achieving them should be considered.
- It is normally the contractor's responsibility to ascertain any apparent or discoverable problems through reasonable investigation and preparation of the works. If a contractor encounters conditions that are unusual, hidden or significantly different from those indicated in the contract documents it is not unreasonable for them to expect additional time or payment to assist in resolution of the issues.

During the course of construction defective or non-compliant work may be identified. It is standard practice to allow the client and/or contractor to order its removal, replacement or repair, but disputes can arise when opinions differ. Such issues are usually resolved during construction, and most defective construction disputes arise after project completion when a constructed element fails. This can be time-consuming and expensive to deal with when the respective design and construction teams have been demobilised, especially if there are allegations of negligence or breach of contract that necessitate insurance indemnification. Thus, dispute avoidance and early resolution should be a priority for all parties.

16.9 Conclusions

An understanding of structural engineering movement and tolerances is fundamental when undertaking all projects, but on occasion there are projects that truly encapsulate these issues and require unique solutions. The new British Antarctic Survey (BAS) Halley Research Station in Antarctica is designed to withstand extreme conditions; high winds, deep snow drifts, extreme minimum temperatures of –50ºC and below, sunless three-month winters, and a site location on a floating ice shelf that moves nearer to the sea every year (see **Figure 16.8**).

The response to these challenges has resulted in a building design that fully responds to its environment. Complete off-site pre-fabrication of the modules was not feasible as the individual components would be unable to maintain the required fit when transported to Antarctica. Each structural steel module frame had to be shipped to site and offloaded onto the sea ice as a complete unit with legs and skis attached so that it could be towed into position. Once in place pre-fabricated floor and cladding panels were then fitted making the task of dealing with aluminium, steel and timber fit tolerances much easier since each material had already acclimatised. In addition, the design used standardised and interchangeable components that made it quicker and simpler to build each module, allowing the BAS to keep a reduced spare parts inventory. The project is quite exceptional, and a fantastic example of movement and tolerances issues taking centre-stage in the design and construction process.

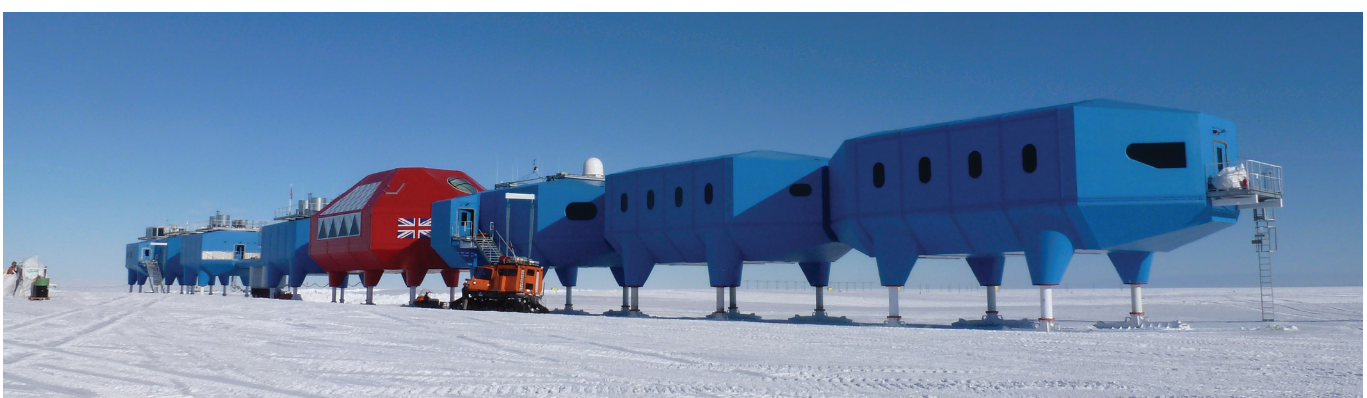

Figure 16.8 British Antarctic Survey Halley Research Station. Design by AECOM and Hugh Broughton Architects (Photo by Hugh Broughton Architects)

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CONTENTS

Chapter 17

Design of concrete elements

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This chapter gives an overview of the design of concrete framed buildings. It will explain the factors to consider in the selection of a particular floor type, including reasons why concrete may be selected ahead of other materials. A number of floor system options are introduced representing the most popular systems in use for situations where labour costs are more significant than material costs. Once a particular system has been selected, there is guidance on how to determine the preliminary sizes for both the floors and the columns. The chapter also explains how to carry out the detailed design of elements for the phenomena covered in Eurocode 2, including flexure, shear and deflection. It explains how to interpret the code for typical elements, providing derived equations and design aids that are not given in the code itself. There is also guidance on determining anchorage and lap lengths.

17.1 Introduction

Concrete is the most widely used construction material in the world: only water is consumed more widely per head of population than concrete. Concrete has been widely used in construction from the mid-nineteenth century, but it was not until the turn of the twentieth century that reinforcement was used to enable concrete to be used as a flexural element. Initially reinforced concrete was a proprietary product with several systems available, but during the twentieth century Codes of Practice were introduced which set out the safe use of reinforced concrete for general use.

This chapter will explain why concrete is used widely in building construction and explain the advantages and design considerations for a variety of floor systems.

The detailed design in the chapter will be based on BS EN1992-1-1 (BSI, 2004), which will be referred to as Eurocode 2. This is the current concrete design standard for the whole of Europe, and increasingly countries outside of Europe. As part of the Eurocode suite of design standards it is considered to be the most advanced concrete standard in the world. However, preliminary sizing rules and general guidance on the use of concrete should be applicable whichever design standard is used.

The other point to note is that the chapter is written with northern European markets in mind. In this region the costs of labour and formwork are high when compared to material costs. It is, therefore, recognised that this may not suit some worldwide markets and local conditions should be considered when proposing initial options.

17.2 System selection

The choice of structural system will be often be determined on the balance of different influences; each construction project is unique and will have a different set of influences. The engineer along with the design team and client will have to assess those options and the benefits. This section is intended to explain the range of influences so that the engineer can make a rational assessment. As this chapter is focused on concrete, the first consideration is why concrete should be used.

17.2.1 Why use concrete?

Concrete is a versatile material, whose raw materials are found throughout the world and therefore in many situations it is the first choice building material. However, the reasons for using concrete go deeper than just easy availability and some of the benefits are explained below.

17.2.1.1 Economy

For the vast majority of building projects, economy is the key driver. The material costs for concrete are generally low compared to other materials, but other factors also have an influence: labour and in particular formwork costs can be up to half the cost of a concrete frame. However, concrete is still an economical material having a market share of more than 50% in most countries, the UK being a notable exception.

17.2.1.2 Programme

In the UK there is a perception that concrete framed buildings are slow to construct. However, this is not necessarily the case. An *in situ* concrete frame construction may take longer than other materials, but overall construction times are comparable. This is because the lead time can be shorter and because follow-on trades can follow more closely behind the concrete works. Further details can be found for various building types in a series of studies by The Concrete Centre (2007, 2008a, 2008b).

Pre-fabrication can also significantly reduce programme times and there are many ways in which concrete elements can be pre-fabricated, including precast columns, twin-wall panels, hollowcore units and lattice girder slabs.

Programme times can also be improved by good detailing and consideration of buildability.

17.2.1.3 Sustainability

Sustainability has become an important factor in the design of framed buildings, and concrete can contribute to a sustainable design, especially when the material properties are understood and used to their maximum. Concrete can contribute to a sustainable building in the following ways:

- Concrete is a local material – reducing travel distances for the materials used.
- Concrete is long lasting – a correctly detailed reinforced concrete building should last comfortably 60 years and would be expected to last considerably longer even with little maintenance.
- Concrete can be exposed – reducing the need for other materials to cover it.
- Concrete is fire resistant – no fire protection materials are required.
- Concrete can be used to minimise energy through the use of its thermal mass – see De Saulles (2006) for more information.
- Concrete can be (and is) recycled at the end of its life.
- Concrete can be used to give flexibility for change of use.
- Materials can be minimised by using prestressing.

17.2.1.4 Fire resistance

Concrete offers good fire resistance because:

- it is a non-combustible material;
- it is a good insulator;
- it has low loss of strength at building fire temperatures;
- strength loss under sustained periods of elevated temperatures is low.

Guidance on ensuring appropriate fire resistance can be found in the section on ‘cover’ below (Section 17.5.1).

17.2.1.5 Acoustics

Concrete is a good insulator of sound, resisting the passage of both impact and airborne sound; this makes it a useful material in those situations where sound reduction between spaces is required and particularly useful for residential, educational and healthcare buildings.

17.2.1.6 Long spans

A common misconception is that concrete is not suitable for long span situations. However, the use of prestressed concrete can greatly enhance the clear spans. prestressed double ‘T’, hollowcore units and post-tensioned *in situ* floors and beams can comfortably span 12 to 16 m – further guidance is given below.

17.2.2 Floor system options

Arguably the most important aspect of a buildable concrete framed building is the choice of floor system, and there is a vast range of options. The section has highlighted the reasons why concrete is widely used. However, there is plenty of scope for innovation and designers should not be constrained by the options presented here. **Table 17.1** provides guidance on the various options available.

Slab type	Description	Advantages	Design considerations
Solid flat slab (Flat plate)	A solid concrete slab of constant thickness supported directly by columns without the use of beams. Widely used because the formwork is simple and therefore cost effective, suitable for spans up to 9 m only	■ Speed of construction ■ Economy ■ Easy services distribution ■ Minimal storey height ■ Easy to install partitions ■ Aesthetically pleasing soffit ■ Inherent robustness	■ Punching shear capacity ■ Deflection ■ Requires continuity ■ Holes should be avoided around columns wherever possible ■ Limited span range
Post-tensioned flat slabs	A prestressed solid concrete slab, supported directly on columns. The prestressing is applied after placing the concrete. The prestressing allows thinner slabs, or longer spans than for a reinforced concrete flat slab	■ Speed of construction ■ Economy ■ Minimises the use of materials ■ Easy services distribution ■ Minimal storey height ■ Easy to install partitions ■ Holes can be formed ■ Aesthetically pleasing soffit ■ Inherent robustness	■ Design is often by specialist ■ Increased shrinkage due to prestress ■ Punching shear capacity is limited
Hollowcore units on beams or walls	Precast, prestressed concrete units spanning in one direction and supported by beams or walls. Voids are created within the units through an extrusion process during manufacture	■ Speed of construction ■ Economy ■ Minimises the use of materials ■ Easy to install partitions ■ Low self-weight ■ Open plan areas ■ Low deflection ■ Off-site manufacture	■ Suppliers use fixed depths for slabs ■ Bedding of the units, especially for long spans ■ Usually supplied with pre-camber, which increases with span

Table 17.1 Concrete floor systems

Slab type	Description	Advantages	Design considerations
Lattice girder slab	A thin precast slab acts as permanent formwork for *in situ* concrete slab. Reinforcement lattice girders are cast into the precast slab to provide the strength in the temporary situation. Can be one-way spanning on to beams, or two-way spanning on to columns	■ Speed of construction ■ Economy ■ Easy to install partitions ■ Holes can be formed ■ Aesthetically pleasing soffit ■ Inherent robustness ■ Off-site construction	■ The joints between the slabs reduce the effective depth at these locations (splice bars are required) ■ Joints should be placed away from locations of high stress ■ Interface shear should be considered between precast and in situ concrete ■ Effective depth for punching shear is reduced, unless shear links are embedded in precast concrete
Biaxial voided slab (trade names: Bubbledeck and Cobiax)	Voids are placed within the slab using hollow spheres of recycled plastic to reduce the self-weight of the slab. Usually used in combination with a lattice girder slab (see above). Can be supported by beams or columns	■ Speed of construction ■ Easy services distribution ■ Void reduce materials used ■ Minimal storey height ■ Easy to install partitions ■ Low self-weight ■ Aesthetically pleasing soffit ■ Inherent robustness ■ Off-site manufacture	■ Currently only available as a proprietary system ■ For use as a flat slab, voids should be omitted around column heads to increase punching shear resistance ■ Design is usual by specialist based on test results
Solid slabs on band beams	A solid slab spanning in one direction on to wide shallow beams. Used to reduce self-weight and increase economic spans	■ Speed of construction ■ Economy ■ Easy services distribution ■ Minimal storey height for longer spans ■ Easy to install partitions ■ Holes can be formed ■ Inherent robustness	■ Band beams can be designed as slab elements (i.e. without shear reinforcement) ■ The band beams can be post-tensioned to maximise spans
Solid slabs on deep beams (or walls)	A solid slab, spanning in one direction only, can be supported on deep beams, but are more often supported by cross-walls. This form of construction is popular in residential type buildings	■ Easy to install partitions ■ Holes can be formed ■ Low self-weight ■ Aesthetically pleasing soffit ■ Inherent robustness ■ Low deflection	■ Can be either cast *in situ* or precast ■ Not very practical for open span spaces
Ribbed slab	A thin slab, supported by a number of ribs, or small downstand beams. Used to reduce self-weight, and formed by placing lightweight formers on the formwork. Historically, clay pots were widely used as permanent formers	■ Holes can be formed ■ Low self-weight ■ Aesthetically pleasing soffit ■ Low deflection ■ Inherent robustness ■ Reduces materials used	■ The ribs span in one direction and are supported by beams, which can be the same depth as the ribs to give a constant depth ■ Slow construction, due to the void formers and placing reinforcement between them
Flat slab with flared column head	A solid concrete slab supported directly by columns which have an increased section dimension immediately below the slab to reduce the punching shear stress	■ Easy services distribution ■ Minimal storey height ■ Easy to install partitions ■ Aesthetically pleasing soffit ■ Open plan areas ■ Inherent robustness	■ Flared heads slows construction (but precast columns could be used) ■ Vertical services distribution adjacent to the column is difficult
Flat slab with drops	A solid concrete slab, supported directly by columns, with a thickened area around the column position primarily to give increased shear strength. Can span further than a flat slab	■ Fairly easy services distribution ■ Minimal storey height ■ Easy to install partitions ■ Holes can be formed ■ Inherent robustness	■ Forming the drops slows construction ■ Drops restrict services distribution
Waffle slab	Voids are introduced into the soffit of the slab, which are known are 'coffers'. These reduce the self-weight of the slab, enabling greater spans than a solid slab. The coffers are usually created using proprietary void formers	■ Easy services distribution ■ Holes can be formed ■ Low self-weight ■ Aesthetically pleasing soffit ■ Inherent robustness ■ Low deflections ■ Reduces materials used	■ Slow construction, due to the void formers and placing reinforcement between them ■ Usually supported directly on columns with coffers omitted immediately adjacent to the columns to increase the shear strength

Table 17.1 Concrete floor systems (cont.)

Slab type	Description	Advantages	Design considerations
Double 'T' units	A thin slab supported in narrow ribs, or beams. Units are usually 2.4 m wide with two ribs per unit. Structurally very efficient for long spans	■ Speed of construction ■ Economy ■ Structurally very efficient ■ Long spans ■ Holes can be formed ■ Low self-weight ■ Aesthetically pleasing soffit ■ Off-site construction	■ Although structurally very efficient, the depth is significant factor for shorter spans ■ Beams required to support the units ■ Usually supplied with pre-camber, which increases with span
Two-way spanning slabs	Solid slabs spanning in two-directions and supported on beams	■ Good for heavy loads ■ Long spans ■ Holes can be formed ■ Low self-weight ■ Inherent robustness ■ Low deflection	■ Often only used for long-span or heavily loaded situations because the formwork is complex

Table 17.1 Concrete floor systems (cont.)

17.3 Preliminary sizing

To enable some initial sizes to be put to concrete members some rules-of-thumb are provided in **Tables 17.2** and **17.3**; the notes to the tables should be read and understood. These rules are only intended to be a quick reference guide and more detailed guidance can be found in *Economic Concrete Framed Elements* (Goodchild *et al*., 2009).

17.4 Stability

The stability of the building is of paramount importance and should be considered at the early stages of a project. Assuming the building does not have a basement or part basement the main considerations for stability are lateral forces which can be caused by:

- wind loads
- geometric imperfections
- accidental loads
- earthquakes.

A concrete framed building should be designed to resist these actions, and this can be achieved either with shear walls or by using a moment resisting frame. Detailed consideration of earthquake loading is not considered here (for a discussion of earthquake loading, see Chapter 10: *Loading*).

Where there are shear walls for lateral stability, they should ideally be arranged so that their shear centre coincides with resultant of the applied forces. In practice this is often not achievable and therefore torsion/twisting moments should be considered. There is also an inherent assumption that the floor acts as a horizontal diaphragm; however, there are occasions when this is not the case: for instance when precast floor units are used without an *in situ* concrete topping, or where there are large openings on a particular floor. The designer should sketch out the load path for all lateral loads from their source down to their reaction in the ground. This will enable potential gaps in the structural resistance to be identified.

A moment frame relies on the stiffness of the connections between the vertical and horizontal members, so a frame with deep beams will have less deflection than one using flat slabs. It is generally considered that three storeys is a sensible maximum height for a moment frame, although clearly this will vary depending on the structural system the floor-to-floor height and the depth of the floor plan (for more discussion on stability, see Chapter 15: *Stability*).

17.5 Detailed design

Before commencing the final design, the structural engineer should ensure that all the design data are place, including:

- fixed geometry for the elements;
- finalised actions on the structure, especially the finishes;
- position of any openings;
- fire resistance periods;
- any particular requirements for serviceability, for example, vibration limits;
- material classes.

The guidance below follows the Eurocode 2 approach which can then be applied as appropriate to various elements. **Figure 17.1** gives the overall design process for an element.

17.5.1 Cover

The cover to the reinforcement is an important aspect of concrete durability and fire resistance, especially when exposed to the elements or for long fire periods. Accordingly Eurocode 2 gives guidance on determining cover for:

- bond
- durability
- fire resistance.

17.5.1.1 Bond

The cover requirements for bond are given in **Table 17.4**.

17.5.1.2 Durability

The minimum cover for durability is denoted $c_{min,dur}$ and can be determined from tables 4.1, 4.3, 4.4 and 4.5 of BS EN1992-1-1. However, in the UK, tables 4.3 to 4.5 should not used and BS 8500-1 (BSI, 2006) should be used instead.

17.5.1.3 Nominal cover

The nominal cover, which should be specified on the drawings, is the minimum cover (the maximum of the bond and durability requirements) plus an allowance in design for deviation (Δc_{dev}). The allowance in design for deviation is effectively a construction tolerance and should be taken as 10 mm, unless measures are taken to ensure that a smaller tolerance will be met.

Floor type	Span range: m	Practical minimum thickness: mm	Span to depth ratio			
			Single span	End bay	Multi-span	Cantilever
Flat slab	6–9	200			23–28	
Post-tensioned flat slab	7–12	200			26–40	
Flat slab with drops	6–12	200			23–30	
Flat slab with flared column heads	6–12	200			21–31	
Waffle slab	6–12	250			14–23	
Biaxial voided slab	7–12	150			20–35	
One-way slab	4–11	150	23–27	27–32		7–10
Beams	–	–	15–20		17–26	6–10
Slab supported by band beams	4–11	150		27–40	33–40	
Band beams	6–12	250	13–24		14–24	
Ribbed slab	6–12	250	17–24		17–27	
Lattice girder slab	4–9	150	20–30	23–35		
Hollowcore slab	4–16	150	24–45			
Precast double tee units	8–16	300	17–26			
Two-way slab	4–12	150	25–34	29–39		–

Notes:
1. The span-to-depth values given encompasses a range of loads from 2.5 kN/m² to 10 kN/m²; a lower value should be used for the higher loads.
2. The span-to-depth values given covers a range of spans; as the span increases a lower value should be used. The combination of high load and long span may be outside the range of span-to-depth values given and a more detailed check should be carried out.
3. For flat slabs, punching shear must also be checked; for small columns in particular, the slab depth may need to be increased to be economic.
4. Hollowcore units are often supplied to fixed depths, which can vary depending on supplier.
5. The use of composite toppings on hollowcore and double units could reduce depth of the units required.

Table 17.2 Preliminary span-to-depth ratios for various floor elements

Percentage of reinforcement	Ultimate axial load: kN								
	1000	1500	2000	3000	4000	5000	6000	8000	10000
1.0	250	300	350	400	475	525	575	650	725
2.0	225	275	325	375	425	475	525	600	675
3.0	225	250	300	350	400	450	500	550	625
4.0	225	250	275	325	375	425	450	525	600

Notes:
1. Suitable for concrete class C30/37 (higher strength will reduce the section sizes).
2. Suitable for internal columns, the following factors can be used to increase the vertical loads:
   - Edge column – 1.5
   - Corner column – 2.0
3. Columns are not slender and are braced (i.e. not suitable for moment frames)

Table 17.3 Preliminary column sizes (mm) (data taken from Goodchild *et al.*, 2009)

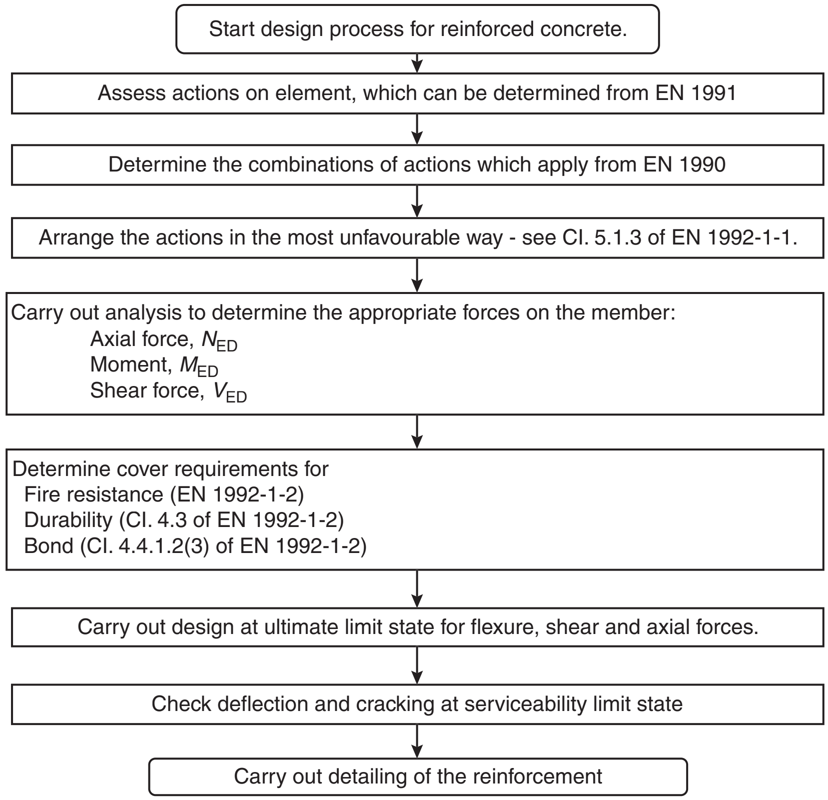


Figure 17.1 Design process for concrete elements

17.5.1.4 Fire resistance

Guidance for fire resistance is provided in a separate standard – BS EN1992-1-2 (BSI, 2004). Although this is an extensive document, covering a variety of approaches, the simplest approach is to use the tabular method in section 5. A variety of tables are provided for a range of concrete element and support conditions. These tables give minimum dimensions and a minimum axis distance. The axis distance is measured from the face of the concrete to the centre of the principal reinforcement, i.e. it is not a cover distance. The relevant axis distance can then be compared with the required nominal cover. For more discussion of fire resistance, please refer to Chapter 11: *Structural fire engineering design*.

Bar/tendon type	Minimum bond requirement, $c_{min,b}$
Single reinforcing bar	Bar diameter, φ
Bundled reinforcing bars	Equivalent bar diameter (See EN1992-1-1, Cl. 8.9.1)
Circular post-tensioning ducts	Duct diameter*
Rectangular post-tensioning ducts	Greater of smaller dimension and half the greater dimension*
Pre-tensioned strand or wire tendon	1.5 times diameter*
Pre-tensioned indented wire tendon	2.5 times diameter*

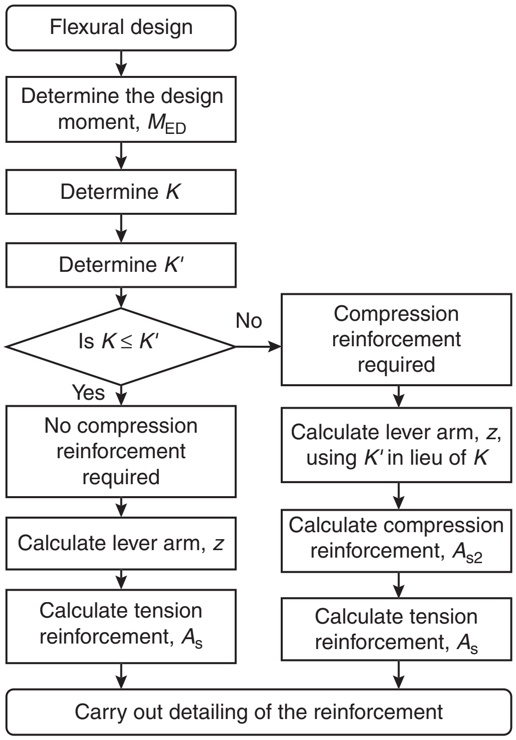
*These values may be amended by a country's National Annex

Table 17.4 Minimum bond requirements

17.5.2 Flexure

The flexural design of reinforced concrete members is well established and is based on assuming that concrete has no tensile capacity. The tension in the section is resisted by the reinforcing steel and the compression by the concrete. Both the concrete and the reinforcement are assumed to act plastically at the ultimate limit state (ULS). A variety of approaches are taken for the shape of the concrete compression zone, commonly referred to as the concrete 'stress block'. The stress block shape can be parabolic, bi-linear or rectangular. Since there is little advantage from use of the more complex stress blocks, generally a rectangular stress block is used. The shape of the stress block may vary between codes of practice. In Eurocode 2 only the stress block limits are presented; designers are expected to work from first principles or turn to textbooks for design equations.

A design process, including design equations, is presented in **Figure 17.2**. In this approach, a limit is placed on the normalised bending resistance, k, to ensure that the reinforcement in the section yields before the concrete crushes. This ensures more ductility close to the ultimate limit state giving more warning of an impending failure. The values given for α_{cc}, k_1, k_2, k_3 and k_4



$$K = \frac{M_{Ed}}{bd^2 f_{ck}}$$

$$K' = \frac{\alpha_{cc}\lambda^2\eta}{\gamma_c k_2^2}\left(\frac{\delta k_2}{\lambda} + k_1\delta - \frac{\delta^2}{2} - \frac{k_1 k_2}{\lambda} - \frac{k_1^2}{2}\right)$$

$$z = \frac{d}{2}\left(1 + \sqrt{1 - \frac{2.5\lambda\gamma_c K}{\eta\alpha_{cc}}}\right)$$

$$A_s = \frac{M_{Ed}}{f_{yd}z}$$ when no compression reinforcement required

$$A_s = \frac{K' f_{ck} bd^2}{f_{yd}z} + A_{s2}$$ when compression reinforcement is required

$$A_s = \frac{(K - K')f_{ck}bd^2}{f_{yd}z(d - d_2)}$$

Where

M_{ED} = Design moment (ultimate limit state)
b = breadth of section
d = effective depth of section
d_2 = effective depth to compression steel
f_{ck} = Characteristic cylinder strenght of concrete
α_{cc} = Coefficient taking account of long term effects on compression (recommended value = 1.0).
λ = factor defining the effective height of the compression zone
= Min(0.8; 0.8 – (f_{ck} – 50)/400)
η = factor defining the effective strenght
= Min(1.0; 1.0 – (f_{ck} – 50)/200)
λ_c = Partial factor for concrete
δ = redistribution ratio
K_1 = factor for $f_{ck} \leq 50$ (recommended value: 0.44)
= K_3 for $f_{ck} > 50$ (recommended value 0.54)
K_2 = factor for $f_{ck} \leq 50$ (recommended value: 1.25(0.6+0.0014/ ε_{cu2}))
= k_4 for $f_{ck} > 50$ (recommended value: 1.25(0.6 + 0.0014/ ε_{cu2}))
ε_{cu2} = Min(0.0035; 0.0026 + 0.035((90- f_{ck})/100)^4)

Figure 17.2 Preliminary span-to-depth ratios for various floor elements

are the recommended values; the UK National Annex has different values.

17.5.3 Axial

Concrete columns are relatively stiff compared to columns in other materials and often the compressive capacity is not limited by the buckling capacity. However, tall columns, columns with small cross-section dimensions or pinned supports can fail due to buckling rather than crushing. Therefore, codes of practice often provide a limit above which second-order moments should be considered. In Eurocode 2, this limit is quite sophisticated, taking into account member geometry, creep, support stiffness, how the moments are applied and the contribution of the reinforcement and is explained below.

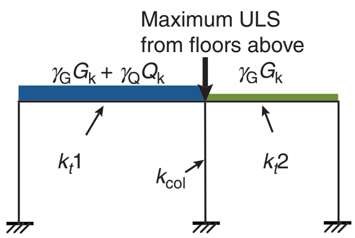
17.5.3.1 Effective length

The effective length (l_0) of a member is used in determination of slenderness. In Eurocode 2, the effective length is defined in Figure 5.7 and expressions (5.15) and (5.16). Unfortunately, in practice it is not simple to apply these in hand calculations for the following reasons. The factor k is used, which requires the rotation of the restraining member to be calculated. k should have a minimum value of 0.1, and for this reason within Figure 5.7, options (b), (c), (d) and (e) cannot be used. Therefore in practice option (f) should be used for braced frames and (g) for sway frames.

An alternative method of calculating k is given in PD 6687 (BSI, 2006) and this can be practicably used in hand calculations. Once k has been calculated at each end of the member, an effective length factor can be calculated or obtained from **Table 17.5** for a braced member.

17.5.3.2 Design moments

The design moments for a column can be obtained from a global analysis, or by a simple hand calculation. The global analysis may also include the effect of geometric imperfections. At a local level, the simplest approach to determine the design moments for a *braced* column is to assume the maximum axial load (ULS) from the floors above. The maximum load is applied to the longest span of beam framing into the column, and on the shorter span the minimum load is applied (remembering in Eurocodes the same partial factor (γ_G) is used for the permanent actions throughout).



A simple sub-frame can be analysed to determine the moment due to actions:

$$\frac{k_{b1}}{2} = \frac{k_{b2}}{2} = \frac{bh^3}{24L}$$

and

$$k_{col} = \frac{bh^3}{12L}$$

where

b = breadth of the member
h = depth of the member
L = length of the member

$$\sum k = \frac{k_{b1}}{2} + \frac{k_{b2}}{2} + 2k_{col}$$

where there is a column above, or

$$\sum k = \frac{k_{b1}}{2} + \frac{k_{b2}}{2} + k_{col}$$

for the column supporting the top storey.
The fixed end moments are calculated for each beam:

$$FEM = \frac{wl^2}{12}$$

The design moment is then

$$M = \frac{k_{col}}{\sum k(b1_{FEM} - b2_{FEM})}$$

At this stage, if they have not already been included, the effects of geometric imperfections should be added to the moment. Using Eurocode 2 terminology this is the moment M_{02}, the largest of the two end moments. $M_{02} = M_{Ed} + N_{Ed}l_0/400$. M_{01} is the smallest of the two end moments.

The minimum design moment in any column section is

$$M_{min} = \min\left(\frac{h}{30};\ 20\ \text{mm}\right) \times N_{Ed}$$

17.5.4 Slenderness

The slenderness for a column section in Eurocode 2 is determined using the radius of gyration and is:

$$\lambda = \frac{l_0}{i}$$

where i = radius of gyration. For a rectangular section this simplifies to $\lambda = 3.46\dfrac{l_0}{h}$

For a braced section, taking default values for the effects of creep and reinforcement, the limiting value of slenderness may be taken as:

k_2	k_1										
	0.10	0.20	0.30	0.40	0.50	0.70	1.00	2.00	5.00	9.00	Pinned
0.10	0.59	0.62	0.64	0.66	0.67	0.69	0.71	0.73	0.75	0.76	0.77
0.20	0.62	0.65	0.68	0.69	0.71	0.73	0.74	0.77	0.79	0.80	0.81
0.30	0.64	0.68	0.70	0.72	0.73	0.75	0.77	0.80	0.82	0.83	0.84
0.40	0.66	0.69	0.72	0.74	0.75	0.77	0.79	0.82	0.84	0.85	0.86
0.50	0.67	0.71	0.73	0.75	0.76	0.78	0.80	0.83	0.86	0.86	0.87
0.70	0.69	0.73	0.75	0.77	0.78	0.80	0.82	0.85	0.88	0.89	0.90
1.00	0.71	0.74	0.77	0.79	0.80	0.82	0.84	0.88	0.90	0.91	0.92
2.00	0.73	0.77	0.80	0.82	0.83	0.85	0.88	0.91	0.93	0.94	0.95
5.00	0.75	0.79	0.82	0.84	0.86	0.88	0.90	0.93	0.96	0.97	0.98
9.00	0.76	0.80	0.83	0.85	0.86	0.89	0.91	0.94	0.97	0.98	0.99
Pinned	0.77	0.81	0.84	0.86	0.87	0.90	0.92	0.95	0.98	0.99	1.00

Table 17.5 Effective length factor for braced buildings

$$\lambda_{\lim} = \frac{15.4\left(1.7 + \frac{M_{01}}{M_{02}}\right)}{\sqrt{\frac{N_{Ed}}{A_c f_{cd}}}}$$

When λ exceeds $\lambda_{\lim}$ the second order moments, M_2 should be calculated and Eurocode 2 gives a number of approaches. The design moment is then:

$$M_{\text{Ed}} = \max(M_{02};\ M_{0\text{e}} + M_2;\ M_{01} + 0.5M_2)$$

where $M_{0\text{e}}$ = the mid-height moment which can be taken as $0.6M_{02} + 0.4M_{01} > 0.4M_{02}$, provided there are no moments applied between the ends of the column.

For a braced column this simplifies to $M_{\text{Ed}} = M_{02}$

17.5.4.1 Column resistance

Figure 6.1 in Eurocode 2 describes the strain limits to be used in determining the resistance of a column section. From this diagram two design equations can be developed: one gives the area of reinforcement required to resist bending, the other the reinforcement required give the column sufficient axial resistance. These two equations are solved iteratively to determine the minimum area of reinforcement required for a particular column. In practice, this is too long-winded for hand calculations and either computer software is used, or column charts are used. Column charts are available from a number of sources, including the website www.eurocode2.info. A design flow chart is provided in **Figure 17.3**.

17.5.5 Shear

Unlike flexural design, there are several theories for determining the shear strength of concrete. One widely used approach is to assumed that the shear strength of concrete comprises a truss (see **Figure 17.4**), with a concrete strut at an angle to the vertical, shear reinforcement acting in tension and the longitudinal reinforcement acting in tension. This basic theory has to

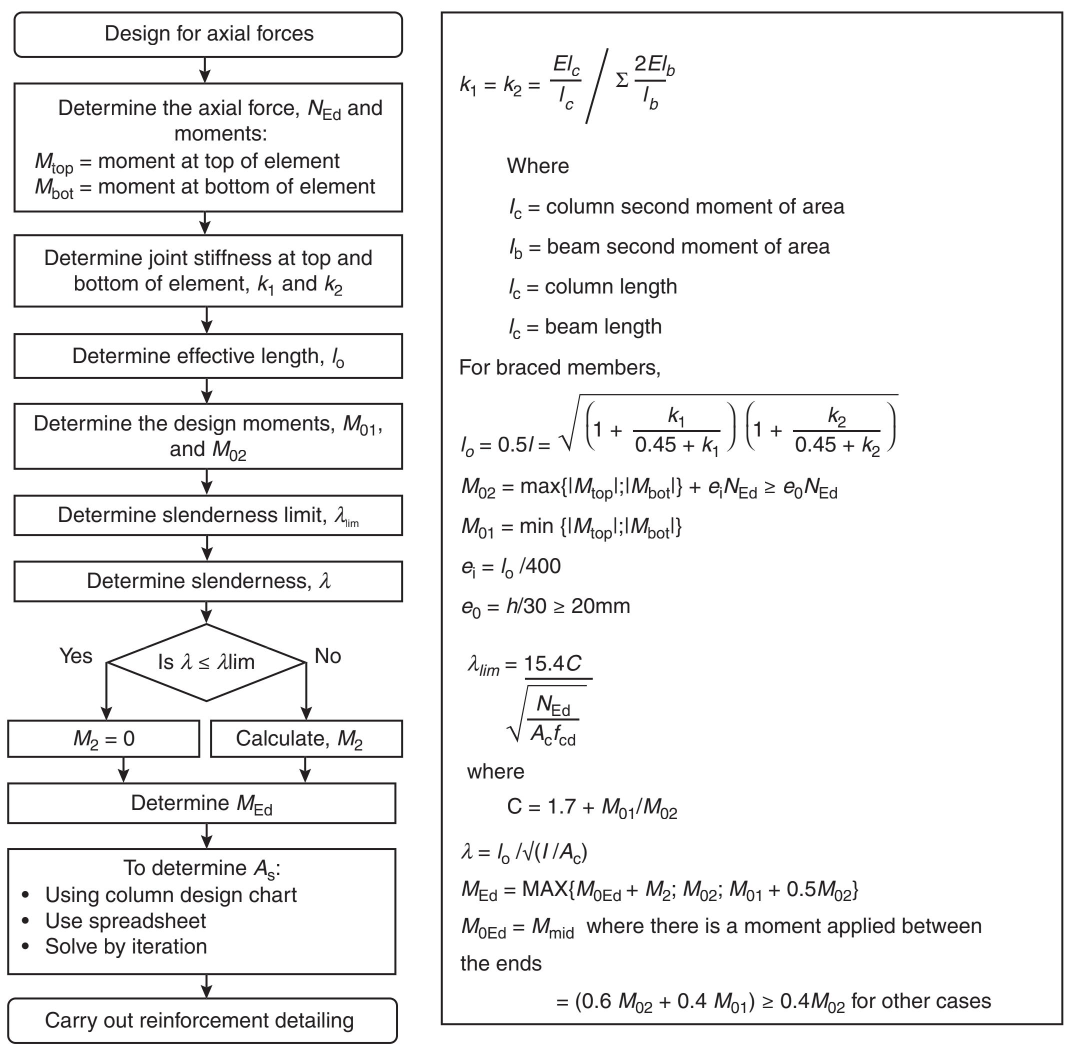


Figure 17.3 Design process braced for axially loaded elements

be empirically modified to agree with the results obtained from experiments.

Where there is no shear reinforcement, the shear strength relies on aggregate interlock and dowel action of the longitudinal reinforcement. For this reason it is recommended that shear reinforcement is always provided, except in members of minor importance. Shear reinforcement may also be omitted in slabs, provided the applied shear force is less than the shear strength of the section without reinforcement.

Punching shear is often critical in flat slabs. Unfortunately, the truss theory described above does not fit with the experimental results for punching shear, and therefore the resistance is based on an empirical approach in Eurocode 2. As a result there are effectively three different approaches to shear design when using Eurocode 2: for beams use the truss methodology, for slabs where shear reinforcement is not required and for punching shear use an empirical approach.

A further note on the shear resistance of concrete is that the depth of the section is an important part of the calculation. This is known as 'size effect' – the deeper the section the lower the shear strength in terms of stress.

17.5.5.1 Beams

In some codes of practice the concrete strut is fixed at 45°. In Eurocode 2, the strut angle can be varied between 22° and 45°; a shallow angle means the concrete strut will cross more shear links and therefore the area of shear reinforcement is reduced – leading to a more efficient design. When designing to Eurocode 2, it will generally be found that a strut angle of 22° will give sufficient shear capacity. It is therefore pragmatic to determine the capacity of the section with a strut at 22° and only if this is insufficient to determine the minimum required strut angle. **Table 17.6** gives the shear stress resistance for various concrete strengths. The design process is provided in **Figure 17.5**.

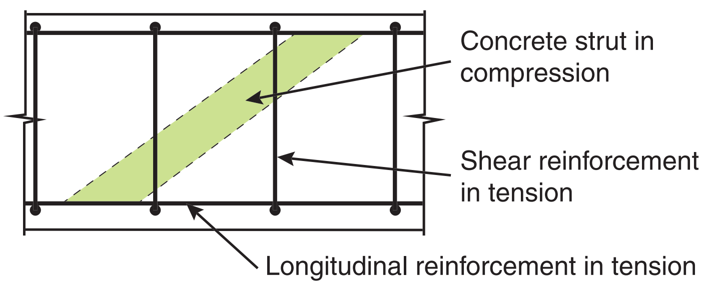
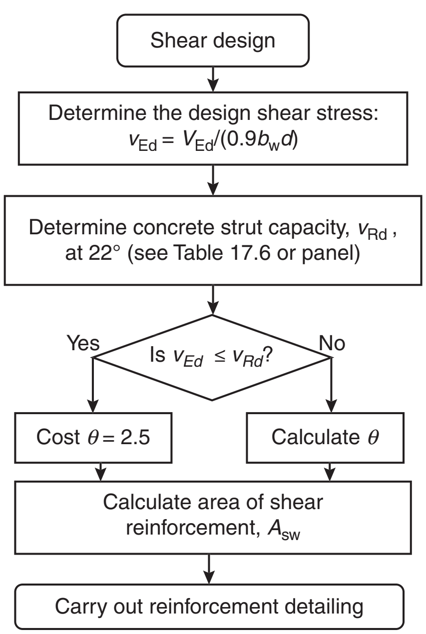


Figure 17.4 Shear resistance using truss theory

f_{ck}	v_{Rd} When cot θ = 2.5 (θ = 22°)	v_{Rd} When cot θ = 1.0 (θ = 45°)
25	3.10	4.50
28	3.43	4.97
30	3.64	5.28
32	3.84	5.58
35	4.15	6.02
40	4.63	6.72
45	5.08	7.38
50	**5.51**	**8.00**

Table 17.6 Values for v_{Rd}



$$v_{Rd} = 0.2\,(1 - f_{ck}/250)\,f_{ck}\sin 2$$

$$v_{Rd} = 0.138\,(1 - f_{ck}/250)\,f_{ck} \text{ when } \theta = 22 \text{ deg}$$

$$v_{Rd} = 0.200\,(1 - f_{ck}/250)\,f_{ck} \text{ when } \theta = 45 \text{ deg}$$

$$\theta = 0.5\,\sin^{-1}\left(\frac{v_{Ed}}{0.20 f_{ck}\left(1 - \frac{f_{ck}}{250}\right)}\right)$$

$$\frac{A_{sw}}{s} = \frac{v_{Ed} b_w}{f_{ywd}\cot\theta}$$

Figure 17.5 Design process for shear reinforcement

17.5.5.2 Slabs

Assuming there is no prestress in the slab, then the shear resistance of the slab can be obtained from **Table 17.7**. This can be compared at the applied design shear stress. The table has been produced for a concrete strength, f_{ck} of 25 MPa; for higher concrete strengths factors are provided at the foot of the table. Designers who are familiar with BS 8110 should note that there is no limit on the maximum effective depth. For depths over 500 mm, the shear resistance, $v_{Rd,c}$ should be calculated using Cl. 6.2.2(1) of Eurocode 2.

17.5.5.3 Punching shear

As has already been seen for column design, as well as vertical loads applied to a column, there will also be a moment. For punching shear calculations this is important as it effectively increase the shear stress in the slab, over part of the shear perimeter. This can be considered by applying a factor, which in Eurocode 2 is β. There are numerous Expressions provided in Eurocode 2 to allow β to be calculated, these include:

- internal rectangular column
- internal circular column
- internal rectangular column with biaxial loading
- edge columns
- corner columns

More simply, for a braced frame where the spans are approximately equal (say within 15% of the longest span), the factors in **Table 17.8** can be used.

The design shear stress is then, $v_{Ed} = \beta V_{Ed}/(u_i d_{eff})$. The shear stress is checked at the column face and then at perimeters around the column. The basic control perimeter is at $2d$ from the column face and for a rectangular column has rounded corners, so that it is always $2d$ from the face of the column. **Figure 17.6** provides a design flow chart for punching shear.

17.5.6 Deflection

Deflection of reinforced concrete is a complex subject; it varies with a whole range of parameters. In essence, the more sophisticated the analysis, the closer to the actual deflection the predication is likely to be. The factors that affect deflection are:

- elastic modulus
- tensile strength
- creep
- loading sequence
- cracking
- shrinkage curvature

Designers should be aware that however sophisticated the analysis, it is the input data that has a significant impact on the predicted deflection. Of greatest significance is the elastic modulus, which is highly dependent on the type of aggregates used. The elastic modulus will vary by ± 25% depending on the aggregate type. Deflection is directly proportional to elastic modulus and therefore the actual deflection can vary by similar percentages. The loading sequence and long-term loading regime are also difficult to predict, but will affect the actual deflection.

In this chapter, only simplified methods will be presented due to space limitations, but further guidance can be found in *How to Design Concrete Structures to Eurocode 2* (Brooker *et al*., 2006). Eurocode 2 has a simplified method based on the use of span-to-effective-depth ratios. Designers should note that unlike some codes of practice, Eurocode 2 allows

$\rho_l = \frac{A_s}{bd}$	Effective depth, d (mm)							
	≤ 200	225	250	275	300	350	400	500
0.25%	0.495	0.474	0.456	0.441	0.428	0.407	0.390	0.365
0.50%	0.557	0.541	0.528	0.516	0.506	0.489	0.475	0.455
0.75%	0.638	0.619	0.604	0.591	0.579	0.560	0.544	0.520
1.00%	0.702	0.682	0.665	0.650	0.637	0.616	0.599	0.573
1.25%	0.756	0.734	0.716	0.700	0.687	0.664	0.645	0.617
1.50%	0.803	0.780	0.761	0.744	0.730	0.705	0.686	0.656
1.75%	0.846	0.821	0.801	0.783	0.768	0.742	0.722	0.690
≥2.00%	0.884	0.859	0.837	0.819	0.803	0.776	0.755	0.722
This table has been prepared for f_{ck} = 25 MPa, for other values use the following:								
f_{ck}	28	30	32	35	40	45	50	
Factor	1.038	1.063	1.086	1.119	1.170	1.216	1.260	

Table 17.7 Values for $v_{Rd,c}$

strength of the concrete to be taken into consideration; as a result designers can reduce predicted deflection by increasing the concrete strength. A flow chart describing the process for checking deflection is presented in **Figure 17.7**.

Where the stress in the reinforcement is relatively low the deflection will be lower, and the modifier given in Expression (7.17) allows for this. Eurocode 2 assumes that the stress in the reinforcement is 310 MPa under quasi-permanent loading. When the stress is below this level, as it will almost invariably be, the basic span-to-effective-depth ratio can be increased. The stress at the serviceability limit state can be calculated from first principles using a triangular stress block for the concrete,

Column Position	Factor, β
Internal	1.15
Edge	1.4
Corner	1.5

Table 17.8 Values for β

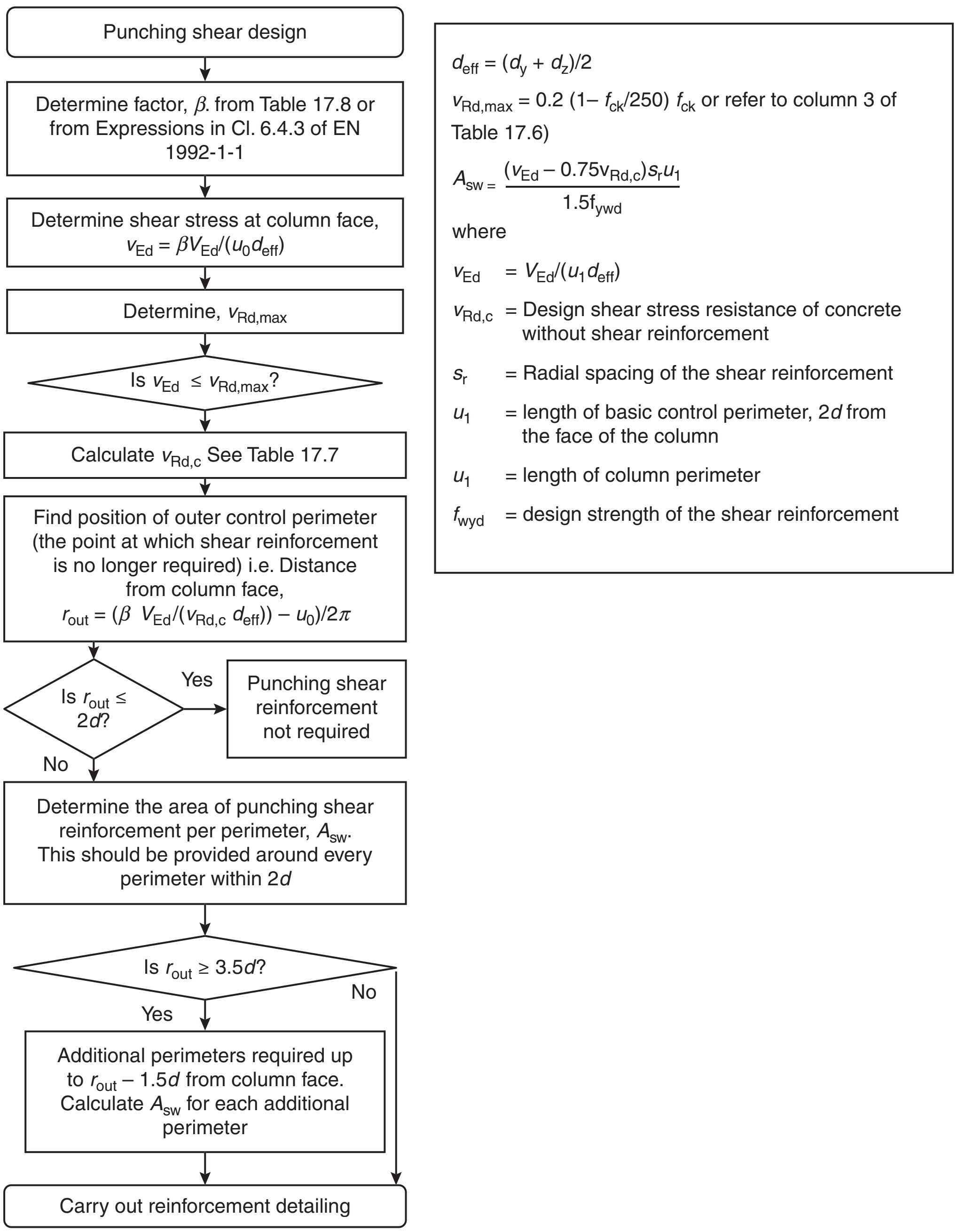
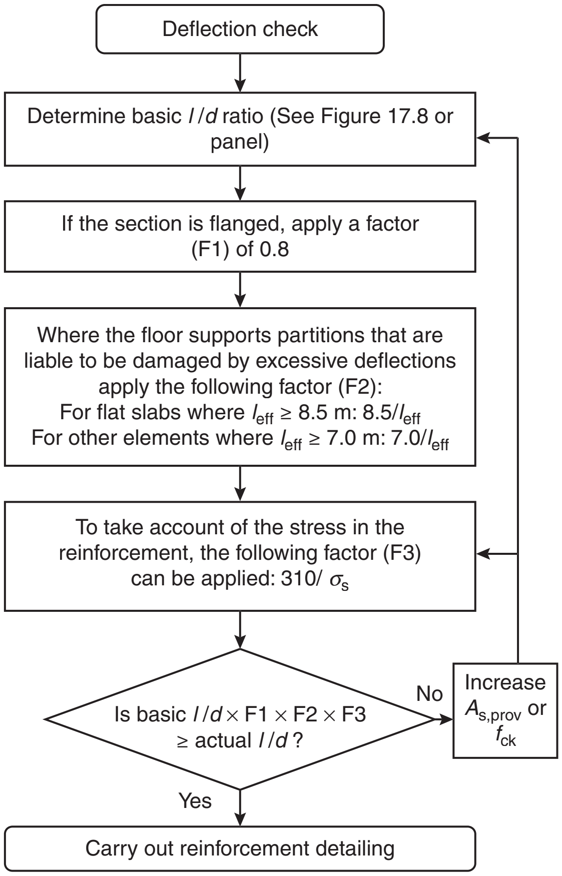


Figure 17.6 Design process for punching shear reinforcement



$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\,\frac{\rho_0}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho}\right)1^{3/2}\right] \text{ if } \rho_0 \le \rho_0$$

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\,\frac{\rho_0}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_0}}\right] \text{ if } \rho > \rho$$

where:
l/d = span/depth ratio
K = factor for different structural systems
= 1.0 for simply supported spans
= 1.2 for flat slabs
= 1.3 for end span condition
= 1.5 for interior spans
ρ_0 = reference reinforcement ratio = $f_{ck} \times 10^{-3}$
ρ = required tension reinforcement ratio to resist the moment due to the design loads
ρ' = required compression reinforcement ratio to resist the moment due to design

The stress in the reinforcement can be estimated from:

$$\sigma_s = \left(\frac{f_{yk}}{\gamma_s}\right)\left(\frac{\Psi_2 Q_k + G_k}{1.5\,Q_k + 1.35 G_k}\right)\left(\frac{A_{s,req}}{A_{s,prov}}\right)\left(\frac{1}{\delta}\right)$$

Figure 17.7 Design process checking deflection

alternatively the following can be used to give a reasonable estimate of the stress at the quasi-permanent limit state:

$$\sigma_s = \left(\frac{f_{yk} A_{s,req}}{\gamma_s A_{s,prov}\delta}\right)\left(\frac{\psi_2 Q_k + G_k}{1.5 Q_k + 1.35 G_k}\right)$$

Designers in the UK should note that the UK National Annex to Eurocode 2 applies restrictions on the use of Expression (7.17) in the form of note 5 to Table NA.5. This notes that σ_s should be calculated for the characteristic action, not quasi-permanent actions (i.e. in the equation above $\psi_2 = 1.0$). In practice, this means that σ_s will always be greater than 310 MPa and only by providing more reinforcement than is required for the ULS will the stress in the reinforcement be reduced sufficiently to reduce deflection. Some would argue that in Eurocode 2 Expression (7.17) is not a Nationally Determined Parameter and therefore the UK National Annex should apply these restrictions in this way.

17.5.7 Cracking

All reinforced concrete will crack; however, the extent of the cracking should be controlled. Limits are usually placed on the size of the crack width, and then reinforcement is designed to limit the cracking. The limits in Eurocode 2 are presented in **Table 17.9**.

Cracking is generally reduced by providing bars at a closer spacing. There are two approaches to the control of cracking: either direct calculation can be undertaken, or simple rules can be applied. Direct calculations should give more accurate results, but just as with deflection there are a number of factors

Exposure class	RC and unbounded pre-stressed members	Bonded pre-stressed members
	Quasi-permanent combinations of actions	Frequent combinations of actions
X0, XC1	0.4[1]	0.2
XC2, XC3, XC4	0.3	0.2[2]
XD1, XD2, XS1, XS2, XS3		Decompression[3]

Notes:
1. For X0 and XC1 exposure conditions, crack wdith does not affect durability and this limit is set for acceptable appearance.
2. Decompression should be checked under quasi-permanent combination of actions.
3. Decompression requires that all parts of the bonded tendons or duct lie at least 25 mm within concrete in compression.

Table 17.9 Crack width limits (mm) (data taken from BSI, 2004)

which influence the cracking and therefore the results are only as accurate as the input data. Eurocode 2 offers some simple rules: either the bar size is limited, or the spacing of the bars is limited (see **Tables 17.10** and **17.11**). It is not necessary to meet both criteria. The stress in the reinforcement should be the stress under quasi-permanent loading and can be estimated from:

$$\sigma_s = \left(\frac{f_{yk}}{\gamma_s}\right)\left(\frac{\psi_2 Q_k + G_k}{1.5Q_k + 1.35G_k}\right)\left(\frac{A_{s,req}}{A_{s,prov}}\right)\left(\frac{1}{\delta}\right)$$

The note in **Table 17.10** (which is identical to that in **Table 17.11**) gives the basis on which the maximum bar diameter or bar spacing are calculated, but it is generally accepted that the tables can be used for typical reinforced concrete elements. Where the element under consideration differs significantly from these values direct calculation should be carried out.

17.5.8 Detailing

Having determined all the element sizes and the reinforcement requirements, the reinforcement needs to be drawn and scheduled, ready for fixing on site. The minimum areas of steel should have been calculated to resist bending and shear. Checks should have been carried out to ensure that deflection is within acceptable limits, but reinforcement is provided for a number of other reasons:

- to control cracking, which occurs due to flexure, shrinkage and thermal effects;
- to support the top layer of reinforcement in slabs and beams;
- to distribute forces into the designed reinforcement, i.e. secondary reinforcement in slabs and walls;
- to prevent buckling of bars in compression, i.e. links in columns.

This additional reinforcement is usually determined through empirical rules, which vary slightly from code to code but are intended to achieve the same end. The empirical rules are usually presented for particular elements, for example, a column, and therefore for some elements that do not fit into these categories, some interpretation is required.

Another important aspect of detailing is determining the anchorage and lap lengths for bars and the position of the laps. A bar should be anchored so that it will not pull out of the concrete under tension or compression. The anchorage can be in the form of a straight length of bar or a through a bend at the end of the bar (subject to limiting rules). Laps are required to transmit tension or compression from one bar to another through the concrete. It is good practice to place the laps at a position of relatively low stress in the bar. For practical purposes the contractor will also want to place day joints around the position of the lap locations.

The Eurocode 2 approach to determining the anchorage and lap lengths is to provide factors that consider the parameters which affect their strength. The bond strength is then multiplied by the factors to determine the appropriate lap or anchorage length. This is at odds with previous practice of using a simple multiple of the bar diameter to give a suitable length. The advantage is that economies of materials can be made by considering each particular situation. The disadvantage is that it makes detailing and fixing more complex. It is, therefore, appropriate to make simplifying assumptions so that standard lap lengths can be used throughout a project. One approach is presented in **Table 17.12**, where a conservative value is given for most of the factors (see the notes), and an appropriate length can be read for a bar diameter for various conditions.

One requirement to be aware of is that the laps should be 'staggered', meaning that the position of laps in adjacent bars should be offset so that high local stresses in the concrete are limited.

17.5.8.1 Information for detailers

The detailing of the reinforcement is usually undertaken by experienced 'detailers' who take the design intention of the engineer and produce the reinforcement drawings and accompanying bar bending schedules. The detailing may be

Steel stress (MPa)	Maximum bar size (mm)		
	w_k = 0.4 mm	w_k = 0.3 mm	w_k = 0.2 mm
160	40	32	25
200	32	25	16
240	20	16	12
280	16	12	8
320	12	10	6
360	10	8	5

Notes: The values in this table are based on the following assumptions: c = 25 mm, $f_{ct,eff}$ = 2.9 MPa, h_{cr} = 0.5h, (h-d) = 0.1h, k_1 = 0.8, k_2 = 0.5, k_c = 0.4, k_4 = 1.0, k_t = 0.4 and k' = 1.0.

Table 17.10 Maximum bar size to control cracking (data taken from BSI, 2004)

Steel stress (MPa)	Maximum bar spacing: mm		
	w_k = 0.4 mm	w_k = 0.3 mm	w_k = 0.2 mm
160	300	300	200
200	300	250	150
240	250	200	100
280	200	150	50
320	150	100	–
360	100	50	–

Notes: The values in this table are based on the following assumptions: c = 25 mm, $f_{ct,eff}$ = 2.9 MPa, h_{cr} = 0.5h, (h-d) = 0.1h, k_1 = 0.8, k_2 = 0.5, k_c = 0.4, k_4 = 1.0, k_t = 0.4 and k' = 1.0.

Table 17.11 Maximum bar spacing to control cracking (data taken from BSI, 2004)

		Bond condition	Bar diameter (mm)								Reinforcement in compression
			8	10	12	16	20	25	32	40	
Anchorage length, l_{bd}	Straight bars	Good	230	320	410	600	780	1010	1300	1760	41φ
		Poor	320	450	580	850	1120	1450	1850	2510	58φ
	Other bars	Good	320	410	490	650	810	1010	1300	1760	41φ
		Poor	460	580	700	930	1160	1450	1850	2510	58φ
Lap length, l_0	33% lapped in one location	Good	260	360	470	690	900	1170	1490	2020	47φ
		Poor	370	520	670	980	1280	1660	2130	2890	67φ
	50% lapped in one location	Good	310	440	570	830	1090	1420	1810	2460	57φ
		Poor	450	630	820	1190	1560	2020	2590	3520	81φ
	100% lapped in one location	Good	340	470	610	890	1170	1520	1940	2640	61φ
		Poor	480	680	870	1270	1670	2170	2770	3770	87φ

Notes:
1. Nominal cover to all sides ≥ 25 mm.
2. Distance between bars ≥ 50 mm.
3. $\alpha_1 = \alpha_3 = \alpha_4 = \alpha_5$ ı= 1.0.
4. Design stress has been taken as 435 MPa. Where the stress is less than 435 MPa, the figures in this table can be factored by $\sigma_{sd}/435$, subject to the minimum lap lengths given in Cl. 8.7.3 of Eurocode 2.
5. The anchorage and lap lengths have been rounded up to the nearest 10 mm.
6. The figures in this table have been prepared for concrete class C25/30, for other concrete class the following factors can be applied:
C30/37 0.89
C35/45 0.80
C40/50 0.73
C50/60 0.63

Table 17.12 Recommended minimum lap and anchorage lengths (mm)

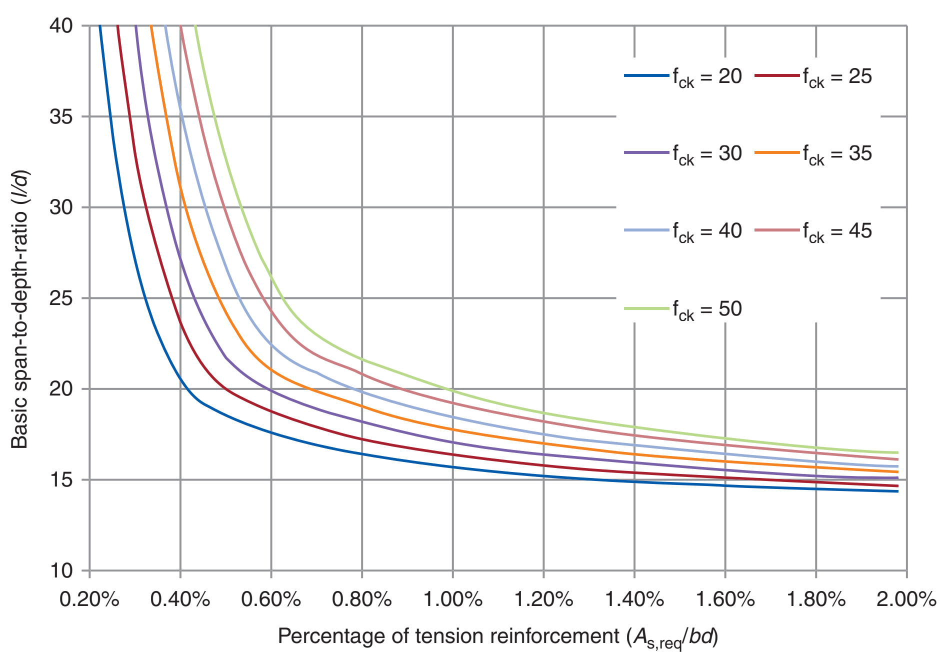


Figure 17.8 Span-to-depth ratio for checking deflection

undertaken in-house, or increasingly out-sourced. It is therefore vital to ensure that all the required design information is presented clearly to avoid errors and misunderstanding and to reduce the number of queries from the detailers. Detailed guidance can be found in *Standard Method of Detailing Structural Concrete* (IStructE, 2006).

17.6 Conclusions

This section on concrete design has provided a brief overview of the approach to using concrete for building structures, from initial through to final design. Concrete is a versatile material that can be formed into many shapes that suit many architectural styles and requirements. It is also a local material with the constituent materials being available throughout the world. Innovations and developments have enabled concrete to remain a competitive structural option, with a variety of choices to suit varying requirements. The use of prestressing can also maximise the potential of concrete by ensuring more of the concrete section is in compression and reducing the volume of reinforcement required.

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17.7.1 Further reading

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Chapter 18
Steelwork

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This chapter outlines an approach to steel design from the viewpoint of a consulting designer practising in the UK. The approach when applied in other countries/markets will apply but should acknowledge local design and construction practice. Rather than a comprehensive guide to steel design this chapter presents summaries, design guidance and sources of information on aspects of design and construction in steelwork with the aim of achieving a successful design. There is a considerable body of published material available, primarily from the Steel Construction Institute (SCI) and the British Constructional Steelwork Association (BCSA). A list of key references and further reading are provided at the end of the chapter.

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CONTENTS

18.1 Introduction to steel design

The designer is often asked to consider whether a steel structure is appropriate when compared with a competing structural material, most likely to be reinforced concrete. The superstructure cost of a single storey building might account for 25%, in multi-storey residential buildings 10–15%, and in non-residential in the region of 10%, but the structural interface with building fabric and services can be a significant cost parameter.

At the outline proposals stage, option appraisals involving column spacing, floor systems and cladding support will normally require liaison with the architect. The building services engineer's input is often required in terms of plant room size, location and the distribution of vertical and horizontal service routes.

When a client or a design team decides that a steel solution is appropriate, the designer may review steel variants as a finer development of the concept appraisal stage. Knowledge of structure cost is needed to assist in the designer's decisions. Section 18.1.7 provides an aid to preliminary steelwork arrangements that would form the initial modelling concepts from which detail design is derived, probably using a 3D analysis, detail design and drawing software. This section is positioned near the end of the chapter in an effort to emphasise that consideration of construction, detailing and cost issues addressed below should precede use of detailed design software or the output is unlikely to provide a successful steel design

18.1.1 Single storey

More than 90% of low-rise single storey buildings in the UK are constructed in steel, most in portal framed steelwork. This contrasts with continental practice where precast concrete competes strongly with steel up to 30 m spans and hybrid frames of concrete columns, glulam beams and deep decking are common relative to the UK. The designer in the UK needs to follow a standard approach of portal framed solutions and associated secondary structures, roofing and walling details to achieve cost effective solutions. Sometimes a portal based solution is not appropriate particularly in mixed storage and office buildings where the office content is above approximately 20%.

18.1.2 Multi-storey commercial

Steel's market share of the competitive steel versus concrete commercial building frame market is close to 70% (2011) in the UK but if spans are less than 9 m concrete will be a strong contender. Structural economy needs balancing with architectural and building services requirements. Lowest cost structural steelwork, 'minimum weight' schemes can sometimes lead to an overall higher cost if interface constraints are ignored.

For example, a main framing grid of 9 m × 9 m, deemed most economic from a structural viewpoint is likely to need secondary steelwork framing to support cladding between the columns spaced at 9 m centres or panelised cladding systems with spanning capability. If perimeter columns are spaced, at 6 m centres, secondary cladding element cost reduces by an order of magnitude compared to any additional cost of structure. It is important to keep in mind the contribution that structure can make in keeping the overall building details cost effective.

18.1.3 Residential

Cold formed lightweight steel sections are extensively used in residential construction integrated into secondary wall and flooring systems or as complete multi-storey framing systems with acoustic and thermal performance requirements integrated in the wall and floor construction build-ups. These cold formed systems follow closely the finishes' build-up and thicknesses of characteristic walling and floor dimensions used in traditional masonry construction.

In some circumstances, hot rolled sections with composite decks and down stand composite beams, or special fabricated sections can be used such as:

- *Slimflor* – where a deck up to 225 mm deep spans onto special shallow fabricated sections providing a minimum floor depth of 300 mm.
- *Slimflor* – where precast slabs span onto special fabricated sections sized to fit within a minimum depth of 200 mm (+tolerance/camber).
- *Slimdek* – where a deck up to 225 mm deep spans onto rolled asymmetrical flanged beams (ASB sections).

18.1.4 Summary, new-build

Structure needs to be appropriate for the application and not necessarily lowest cost. The initial steel frame design is often costed more accurately than the stage of building detail development justifies. Building details need developing with the other team members. If there is inadequate time for development of design details, cost provisions for details to be resolved should be incorporated in the initial estimate.

The challenge for the designer is to avoid the temptation to over-optimise structure. **Figure 18.1** shows an example of a single column used at an expansion joint position that led to a severely compromised detail instead of simply using a pair of columns.

Hollow sections are more costly than open sections but their use can provide benefits in detailing, durability and cost as shown in **Figure 18.2**.

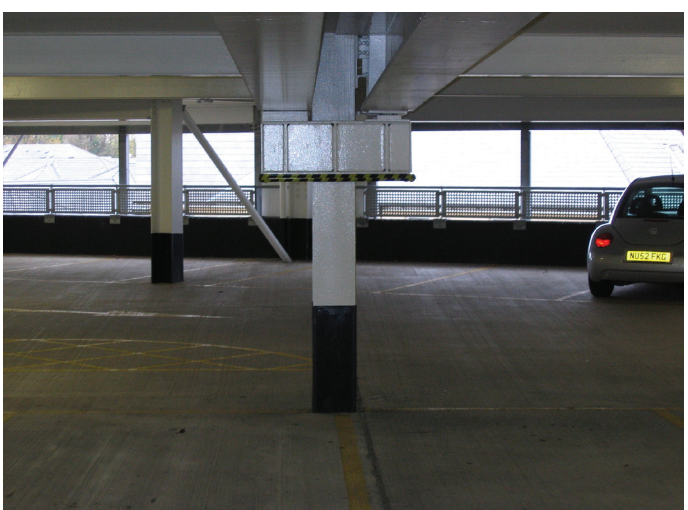

Figure 18.1 An ill-thought-out connection at an expansion joint compromising headroom. A twin column arrangement was the obvious cost effective and structurally better solution

Figure 18.2 Cost-effective detailing exploiting hollow sections, simple joints and corrosion addressed with galvanising

Secondary cladding elements often use aluminium or stainless steel, which are costly relative to steel. When included in cost planning rates used for elevations/cladding there may not be adequate provision for interfacing with a 'low cost' structure. An unexpected cost penalty might result on the overall project. Alternatively, if cladding costs do not incorporate secondary structure, the provision of 'secondary steelwork', identified late, is sometimes added to the mainframe costs causing a cost overrun on the structural costs. Ambiguity is best avoided by clarifying scope and cost of secondary steelwork with generic cladding details and spans defined at early costing stage or by making cost provision for design development risk.

The structural designer needs to be active in the development of construction details at an early stage. Forming the contextual background to the structural designer's choice of section sizes and design topology is an important task communicated by details, drawings and written structural philosophy statements. Designers typically estimate that 5–8% of the overall steel member weight is adequate provision for connections. Some plan arrangements should signal potential problems with complex costly arrangements and be avoided. For example, **Figure 18.3** shows an eight-member junction at a shallow roof apex. If architectural roof detailing had been more practical using either a flat roof and high performance membrane or simple roof falls this might have been avoided. The risk of unexpected costs emerging as the design develops can thus be reduced.

The designer needs to be conversant with the advantages and disadvantages of steel construction. Steel is unaffected by creep, shrinkage, pre-stress losses or time-dependent loading. This makes deflection prediction straightforward which is particularly useful where transfer structures are required.

However, steel structure is relatively low mass. This requires attention to deflection compatibility with finishes and satisfactory dynamic performance.

Figure 18.3 At concept stage, avoid multiple member junctions

There is an increased demand for mixed use and naturally ventilated buildings. Often hybrid structural combinations that exploit particular characteristics of the structural material are used as shown in **Figures 18.7(a,b)**, **18.9**, **18.10(a,b)** and **18.11**.

Steel internal columns are compact relative to concrete columns and can be used to maximise internal usable space if compact column casings are used. On perimeters, steel columns integrated into the wall construction achieve a flush internal finish benefiting internal space planning and rents based on net floor area as shown in **Figures 18.4**, **18.5**, **18.6**, **18.8** and **18.9**.

Flush concrete soffits have become more popular and while steel columns can be integrated into concrete flat slabs to useful effect as shown in **Figures 18.7(a, b)** it would be more usual to use slimflor construction as a competing solution to *in situ* concrete albeit with bottom steel flanges visible, as shown in **Figures 18.10(a,b)** and **18.11**.

Hybrid buildings require the designer to manage the design and detail interfaces of dissimilar materials sometimes involving different contractors. Familiarity with general building construction, appreciation of structural design parameters and frame connection detailing, and an appreciation of overall stability and the interfaces of design responsibility are needed.

It is worth noting a designer's responsibility for overall stability as defined in the UK structural codes. CL 2.1.1.2 from BS 5950-2000 states:

> Overall stability: The designer who is responsible for the overall stability of the structure should be clearly identified. This designer should ensure the compatibility of the structural design and detailing between all those structural parts and components that are required for overall stability, even if some or all of the structural design and detailing of those structural parts and components is carried out by another designer.

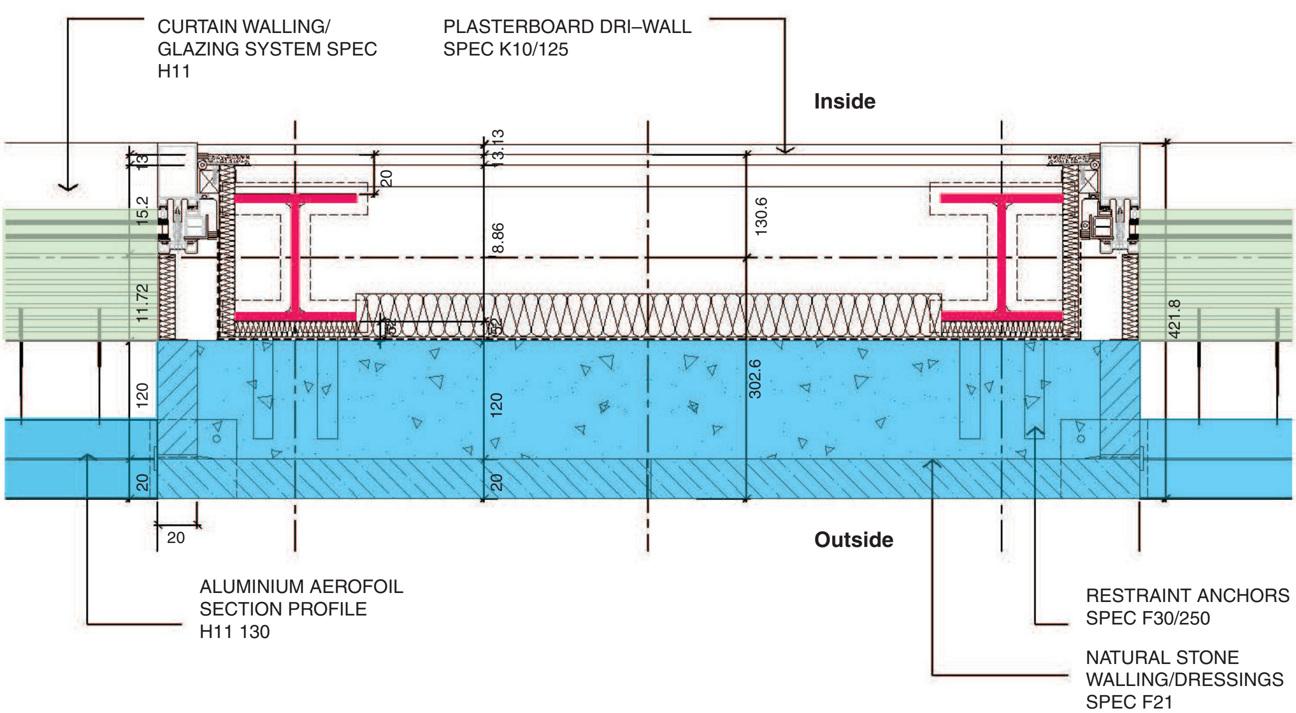

Figure 18.4 Plan of twin perimeter columns set into the wall construction, galvanised as they are in contact with the concrete cladding

Figure 18.5 Elevation of twin perimeter columns set into the wall construction

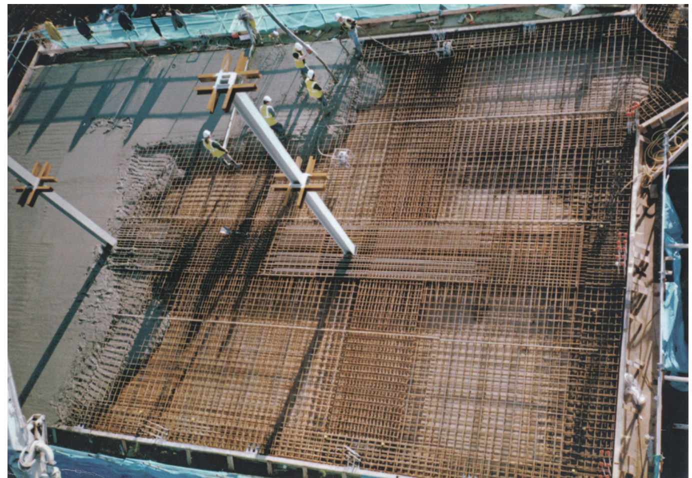

Figure 18.7(a) Steel columns integrated into flat slab concrete construction

Figure 18.6 Flush internal space

Figure 18.7(b) Steel columns integrated into flat slab concrete construction

On the same subject, the Council of American Structural Engineers (CASE) provides a more detailed explanation of the engineer's roles on a building project. It can be assumed that the roles described will be applicable to engineering design worldwide. According to the CASE website:

> The Structural Engineer of Record (SER) perform[s] or supervise[s] the analysis, design, and document preparation for the building structure and has knowledge of the requirements for the load carrying structural system. The SER is responsible for the design of the primary structural system, which is the completed combination of elements which serve to support the buildings self-weight, the applicable live load which is based upon the occupancy and use of the spaces, [and] the environmental loads such as wind, seismic, and thermal.
>
> A Specialty Structural Engineer (SSE) performs structural engineering functions necessary for the structure to be completed and is someone who has shown experience and/or training in the specific specialty. The SSE is usually retained by a supplier or subcontractor who is responsible for the design, fabrication, and (sometimes) installation of engineered elements or by the general contractor or subcontractor(s) responsible for construction related services. Common examples of such elements are precast or tilt-up concrete, open web steel joists, pre-engineered cold-formed steel or wood trusses, and metal building systems.

The CASE website goes on to explain that every project should have a single designated SER who establishes the structural design criteria and concepts for the project, consistent with the role described in the UK codes. The SER may delegate

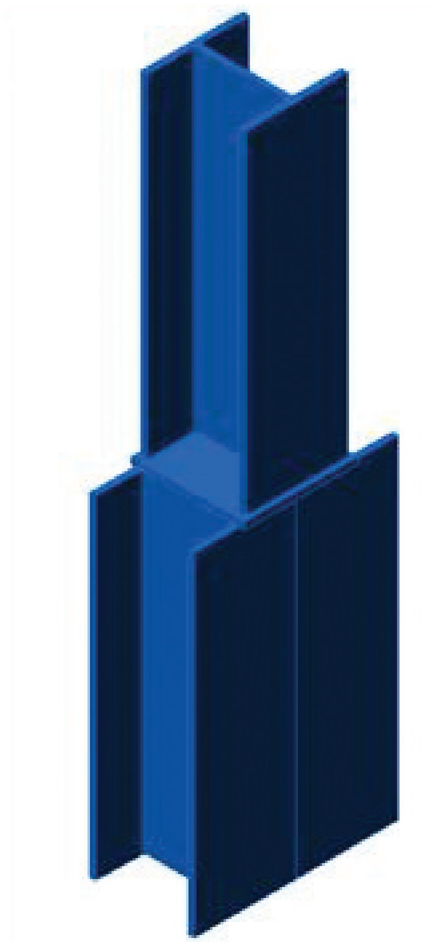

Figure 18.8 Possible perimeter column splice to achieve flush internal space, see circle in Figure 18.9

Figure 18.9 Elevation on perimeter column splice

the detailed design of certain portions to the SSE by communicating this information and other requirements on the construction documents (drawings and specifications). In such cases, the SSEs subsequently prepare calculations and construction documents of their own for the delegated work and submit them to the SER, who verifies that they comply with the specified requirements and are consistent with the project as a whole. This procedure applies directly to typical practice in steelwork design where the steelwork contractor details connections.

18.1.5 Refurbishment

The life of a building structure might easily be in excess of 60 years. Age has no adverse impact on steelwork provided it is protected from corrosion. Its structural properties are unaffected by time-related change such as creep, shrinkage or chemical change that might affect durability and strength in other materials.

It is common to see significant refurbishment due to change of use within a 20-year period. The longevity of steel framed structures means structural modification is an important part of a designer's work. From the mid-twentieth century modern steel design and construction was in use and will be familiar to practising designers. In addition the scale and growth in industrialised economies over the nineteenth and early twentieth centuries provides a rich heritage of iron and steel structure buildings still in use. The trend for regeneration and conservation means a designer is now likely to face the challenge of dealing with buildings incorporating cast iron, wrought iron and early steel.

An outline knowledge of the history of development and construction practice for iron and steel in buildings in Britain helps in appreciating how design parameters that affect modern framed designs have developed. Early building structures were hybrids of load-bearing masonry walls and internal timber framing that incorporated iron in beams and columns.

18.2 History

Record drawings may be available but these can only be regarded as preliminary – pending independent checks that a designer should carry out to validate assumptions.

Figure 18.10(a) Parallel continuous UC sections (concrete infill between) used with precast planks

Figure 18.11 UC sections used with precast planks

Figure 18.10(b) Parallel continuous UC sections (concrete infill between) used with precast planks

18.2.1 Cast iron

Large-scale use of iron in buildings depended on a series of eighteenth-century developments in the fuelling and smelting of iron ore, coal/coke replacing wood/charcoal and advances in furnace design leading to the production of cast iron containing less carbon.

Sulphur in coal led to crumbling of the iron during forging. In 1709 Abraham Darby overcame this problem by using partially burnt coal as coke. Later developments included Henry Corts' reverberatory furnace in 1784, which increased the amount of carbon removed from iron by enhancing oxygen flow across the melt. This made it more suitable to work into wrought iron.

Serious fires in timber floored industrial buildings encouraged the use 'fire resistant' flooring and cast iron structure. The earliest fully framed building in the UK was the Ditherington flax mill built in Shrewsbury in 1797 with cast iron cruciform and tubular columns and inverted 'Y' beams used with brick jack arches providing fire resistance to the sides of the beams.

Re-melting pig iron, which typically contained 4% carbon, in the cupola furnace allowed manufacture of castings to develop on an industrial scale. The decorative potential of cast iron played an important part in its development. Notable examples of framed and panelled structural facades exist in America. In the first half of the nineteenth century, cast iron spans of approximately 25 feet (8 m) were common. **Figure 18.12** shows an 11 m example in a building adjacent to a small London gasworks decommissioned in the 1860s, possibly used for coal storage.

18.2.2 Wrought iron

Well-publicised failures demonstrated the tensile and manufacturing limitations of cast iron. In 1851, The *Civil Designer and Architects Journal* was urging 'the expediency nay the necessity, of employing wrought or rolled iron, instead of cast iron girders'. Advances in hammering and rolling in the 1850s made wrought iron economic. The development of more

Figure 18.12 1850s cast iron column and 11 m beams (note probable nineteenth-century attempt to strengthen the beams with tie rods)

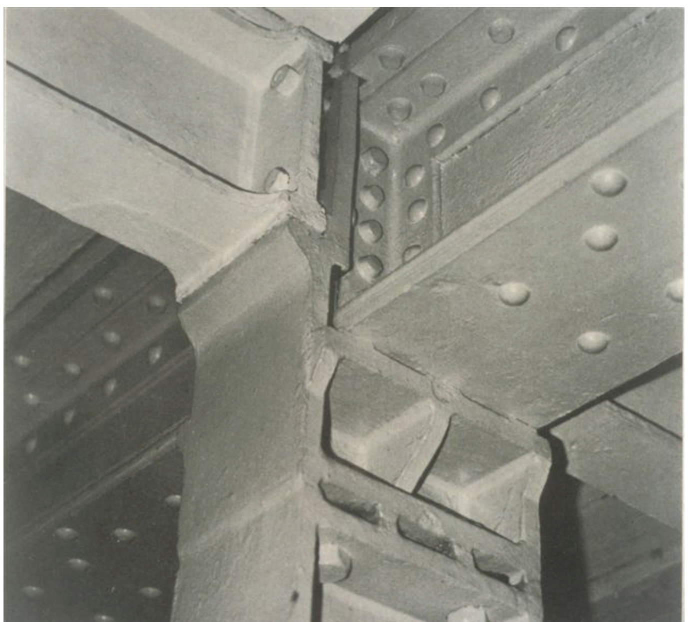

Figure 18.13 Sheerness connection (note land on beam to cast iron column detail providing the frame stiffness, shallow cast iron beam to the left and riveted wrought iron main beam to the right)

efficient processes for wrought iron facilitated the development of bulk steel production.

Wrought iron was produced either by direct reduction of iron ore by heating with charcoal or by heating cast iron in the presence of oxygen. When worked, wrought iron gains strength as carbon content reduces to 0–0.2%. The cost of wrought iron meant that small sections were produced, typically up to 200 mm deep. Larger girders made from plates, flats and angles were riveted into compound girders.

Early iron framed single-storey structure was used in glasshouses, the Crystal Palace of 1851 being a notable example. In 1858, the Boat Store was built at Sheerness (see **Figures 18.13** and **18.14**). The first multi-storey framed building in the world, it has cast iron I-section columns and beams and riveted wrought iron plate girders. Structural I-sections were rare prior to the bulk production of steel in the 1880s when hot rolling became the favoured manufacturing method.

Figure 18.14 Boat Store Sheerness 1858

18.2.3 Early steel

Britain and Belgium were the foremost users of iron in 1850. In 1870, Britain was still producing half the world output of iron at 6 million tons but less than 10% emerged as steel. Bulk production of steel became possible following the invention of the Bessemer converter and the open-hearth furnace in the 1850s/60s. By the end of the century, steel had replaced wrought iron (see **Figure 18.15** for a summary of the change in use of material).

Construction in steel emerged in the latter part of the nineteenth century. In 1896 the first British steel framed building was built in Hartlepool. Some overlap exists in the use of wrought and cast iron: the latter was occasionally used up to the 1920s in columns.

Rolled steel I-beams, channels and angle sections were produced by various manufacturers from about 1883. The British Constructional Steelwork Association's *Historical Structural Steelwork Handbook* (BCSA, 1989) includes tables of steel sections from 1887 onwards.

Standardisation of steel sections in the UK began with the publication of BS 1 in 1901 followed in 1903 by the first edition of BS 4, for structural steel sections listing beam depths from 3 in (76 mm) up to 24 in (610 mm), known as British Standard Beams (BSB). Broad flange beams initially imported from Luxembourg in 1902 were precursors of the Universal Column section. These sections were ideal as beams for minimal headroom applications and for columns occupying a compact 'footprint'. By the 1930s large broad flange beams were being rolled, up to 40 × 12 in (1016 × 305 mm) at 234 lb/ft (348 kg/m). Later developments saw the introduction of New British Standard Beams and New British Standard Heavy Beams (1921), a replacement range of BSB (1932), and finally the Universal Beams and Universal Columns we know today (1959).

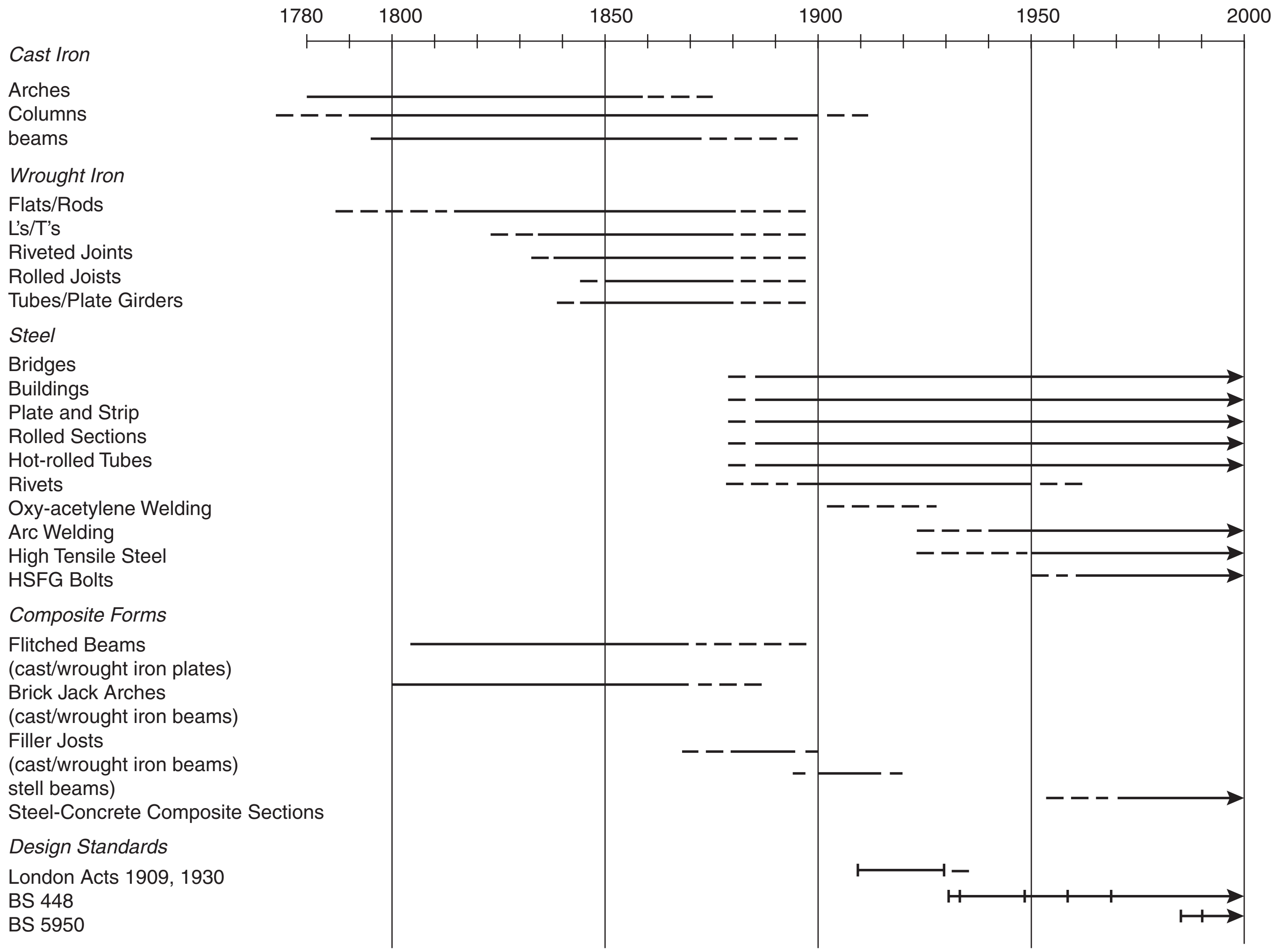

Figure 18.15 Main periods of cast wrought iron and steel (reproduced from SCI 138 (1997) with permission)

18.2.4 Reconstruction of a 1920s steel framed office building

A case study of the reconstruction of 68 King William St, London is included later in this chapter. This provides guidance on a challenging topic that will become more important with the increasing trend for refurbishment.

18.3 What is a successful steel design?

Each party in the design team will have a different perception of what constitutes success. An analyst will regard the practical realisation on site of a complex 3D computer model as a success. A designer will be content in the knowledge that the frame he or she designed met the architectural parameters and the client's budgetary constraints. A main contractor will regard a frame delivered on time with limited variations and meeting initial cost allowance as a success. A steelwork contractor will be concerned to source, in time, the different section sizes required and to adjust a design to suit the preferred manufacture/construction. A subcontracted erector would be interested in keeping the erecting gang safe and busy with deliveries coming to site to suit the erection programme. The erector may be sharing craneage so main contractor relations need to run smoothly.

Things can go wrong and delays to site progress tend to be particularly risky in terms of the costs claimed because of design inadequacies. Some examples of projects that have led to litigation in the UK signal possible threats to successful steel construction that are equally applicable to steelwork construction in other countries.

Phrases used in a design office that should sound warnings include:

- 'The steelwork contractor is designing all the connections, we do not need to consider their design.'
- 'We just design foundations, the steelwork contractor is designing everything else.'

- 'Secondary steelwork is by others, it is probably in the QS's cladding cost provision. It will be sorted out by the architect.'
- 'Access, site conditions, propping and erection will be sorted out by the contractor.'

18.3.1 Case 1

The steelwork contractor's claim for the costs of plating and strengthening beams and columns 1–1.5 m out from each connection was refused, leading to litigation. The designer had changed the frame's stability concept from 'simple' to 'rigid'. The 8-storey frame was a 'minimum weight' design with a column section change at each floor. There was no reserve of bending strength in the beams and columns, and the selected sections were unable to sustain the bending moments without extensive modification. The steelwork contractor won the argument that the tendered design required a price for 'simple connections' and could not be considered to include costs for rigid connections.

The message here, is that risk of compartmentalisation of design, between connections and frame member selection column, and beam selection and the need to provide a design consistent with pricing information, needs to be monitored during design development. The designer needs to retain control of the frame design philosophy in most building types (see section 18.1.4 describing CASE's view of the roles of the engineer). Exceptions may be in single-storey construction where the overall engineer may be the steelwork contractor.

18.3.2 Case 2

The main contractor involved in a design and build contract had unexpected costs to resolve the dimensional coordination of staircases in a multi-storey frame. Escape widths specified on the architect's layouts could not be accommodated between columns that had heavy externally plated column splices not anticipated by the team until after the frame was constructed. The splices had to be cut back and the column flanges site butt-welded to avoid increasing the outside dimensions of the columns. Other site modifications were needed to resolve late coordination problems including doors clashing with bracing, inadequate headroom, and additional steelwork needed on site to suit the architect's layouts issued late in the programme and not reflected on the structural drawings that were issued early in the programme for steelwork manufacture.

The message here is to be aware of the support other team members can anticipate from the structural designer during design development. Initial structural design information issued for tender/pricing without proper guidance on the allowances for design development is common. The steel contractor may be involved late in the design programme and required to price information not knowing it might be partially coordinated. Resolution of structural details such as column splices and their interface with architectural requirements is the designer's role. Guidance should be provided on the drawings and in specifications if unresolved details exist which are likely to compromise architectural criteria or to have cost implications.

18.3.3 Case 3

A waste transfer station had refuse containment walling connected to the portal frame columns. In addition to unanticipated impact load from front end loaders, the frame sway deflections were excessive due to a combination of column under-sizing and frame assumptions being detailed designed by the steelwork contractor for rigid based fixity. Neither were provided in the foundation design by the designer.

The message here is that the designer needs to properly understand the client's intended use and provide adequate definition of frame design parameters even if parts of the structural design are by others.

18.3.4 Case 4

Universal beam (UB) sections used to support brickwork over openings twisted during wall construction, leading to postponement of the work pending redesign and site work including section stiffening, connection strengthening and in some instances replacement with hollow box sections. The main design and build (D&B) contractor claimed delay and repair costs.

The message here is that the designer needs to be alert to torsion control of steelwork in critical locations such as perimeter support of cladding. Closed hollow sections are often the most effective solution but the material cost is approximately double open section costs. While this might discourage their use the cost of open sections with stiffening often proves more costly particularly when the need is identified late in the programme. Lack of resolution of so-called 'secondary' steelwork is a common cause of unsuccessful steel construction. The designer should provide weight and/or cost estimates if not clearly identified in tender/pricing information.

18.3.5 Case 5

Concreting of a composite deck forming an industrial mezzanine floor was abandoned when efforts to level the slab led to progressive deflection of the floor beams under load from the concrete placing operation. The composite floor beams had been designed to be propped during construction but there was no construction guidance on the designer's drawings.

It is not unusual for the temporary support required by the design assumptions not to be transferred from design software to the pricing and construction information. If the designer retains control of the frame design philosophy as described by CASE (Section 18.1.4) above, this will improve communication of key construction information. Construction (Design and Management) (CDM) requirements (HSE, 2007) apply in the UK for the designer to communicate construction methods. **Figure 18.16** shows the temporary support arrangement needed to erect a frame requiring support of beam ends prior to site welding. Other common situations requiring temporary stability of steelwork are long span roof beams with long span

Figure 18.16 Temporary works integrated into the permanent works can be cost effective solutions to high level access – in this case a site weld 15 m above ground level

structural decking. The designer needs to show the temporary steelwork required for stable and safe building assembly.

18.4 Design responsibility

Clarity of design responsibility is essential for successful steel construction and needs to draw on the skills of both the designer and the steelwork contractor with main contractor programme and delivery parameters that can often dictate design decisions. There is also a need, to acknowledge change management and the fact that structural economy must satisfy aspects of factory fabrication, delivery and erection on site.

A separate section follows below addressing steelwork cost but the designer/designer's management of dimensional coordination, design development, change and design team liaison with the steelwork contractor is a pivotal role. This is consistent with the designer's responsibility for overall stability set out in Section 18.1.4 above and the reader is directed to the comments on roles by the CASE extracts in that section. In short, while reference is made to UK guidance the issues addressed are applicable to steelwork construction generally.

18.4.1 Designer appointments

This section touches on legal responsibility because steelwork procurement and construction has particular challenges to address due to off-site manufacture and the need to limit/avoid site modification. There are generic documents for UK practice such as the New Engineering Contract (NEC), GC/Works/5 and PPC2000 that integrate a design team's formal terms, conditions and scope of services with the contractor's design responsibilities and in particular who leads design coordination and change management.

However, it is more common for designers' responsibilities to be defined in 'stand alone' documents, often not formally integrated into the lead designer's responsibilities, normally an architect; or the contractor's/subcontractor's design duties. Responsibilities can also become compartmentalised in 'team' matrices. Design compartmentalisation increases risks to safety (see *CROSS Newsletter* No. 20, October 2010), and there is an associated commercial risk if coordination is not effective.

18.4.2 Legal/commercial context

The designer needs to be alert to his or her duties in law, in the UK CDM 2007 Regulation clause 11, and the duties inferred from the agreement with the client. Agreements often have scopes of services similar to those defined by, for example, the Association of Consulting Designers' *ACE Schedule of Services – Part G (a), 2009* (ACE, 2009). Pages 5–7 of that document require:

- Information for an outline cost plan.
- Integration into the design of requirements of specialist sub-consultants, contractors or sub-contractors. (See CASE's comments in Section 18.1.4 above. **Figure 18.17** shows lightweight steel construction by a specialist that would not normally be detailed by the overall designer.)
- Assistance in coordinating the overall design.
- Tender documentation.
- Calculations and details required for submission to statutory authorities including coordination of specialist supplier/contractor information.
- Advice on appropriate forms of contract, tender invitations and relative merits of tenders, prices and estimates.
- Review of detailed designs, shop fabrication drawings, standard details, and specifications submitted by contractors for conformity with the designer's design and in particular general dimensions, structural adequacy of members and connections and compliance with performance criteria.

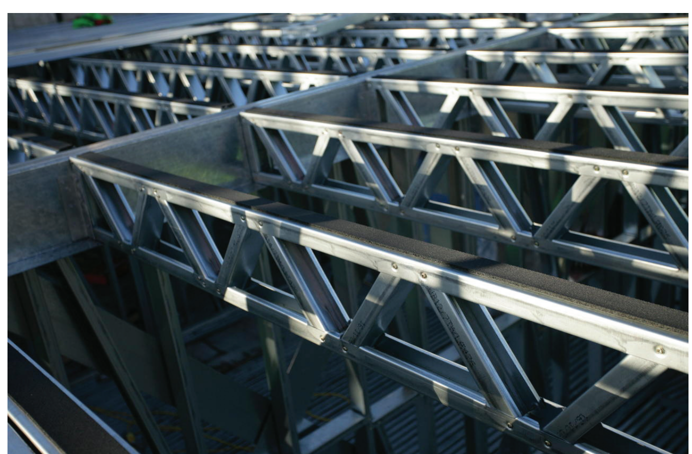

Figure 18.17 Coldformed Metek framing; some steel solutions will be specialist designed

18.4.3 Industry guidance

Traditionally UK codes provided a mix of design and practice guidance. As Eurocode use becomes more common, good practice guidance produced by the steel industry will be relied upon. Not only in the UK but also worldwide there is an ongoing effort to encourage an orderly definition of roles, for example, in the advice from CASE reproduced in Section 18.1.4 above. In 1995/7, a 'Eureka' project (SCI, 1995 and 1997) aimed to raise awareness of the influence design decisions have on the overall buildability and cost of a steel framed building and how to avoid conflict in the design and construction process, reducing the likelihood of expensive remedial site work. Six points were identified that the designer 'must address':

- Recognise the complexity of the design process.
- Establish an appropriate design team.
- Agree information and programme.
- Coordinate contributions.
- Manage the interfaces.
- Control design development.

There is no substitute for the steelwork designer to prepare a project-specific interpretation addressing the duties summarised above, perhaps by way of a project execution plan or notes. In summary, the designer must address commercial requirements to achieve a successful design in steelwork.

18.5 Design, analysis, detail design – a virtuous circle

Perhaps a way to describe the complex transition of concept through to construction is in terms of a 'virtuous circle' of design, analysis and detail design. Detail considerations precede analysis and involve key interface requirements on the structure. Architectural, building services and secondary structural details are reflected in the way the model is imagined and analysed, whether by computer or traditional hand-calculated model approximations. Analysis and modelling tools are in general use and guidance on 3D framing arrangements and standard cross-sections for floor, wall and roof constructions are widely available.

However, modern steel construction practice tends to compartmentalise steelwork contractors and consultant designers. Steel detailing neglected by consultant designers and left to the steelwork contractor to detail for manufacture and erection, means that member arrangements may be derived that do not exploit the geometrical characteristics of steel material to benefit fabrication and buildability on site. Made in a factory, using sections delivered from a steel mill and erected at a probable rate of 80–100 tonnes a week a modest frame can be erected in a month. The designer may go to site but it is likely that the designer has never been inside a fabrication shop and is even less likely to have seen the manufacture and rolling process.

Decisions on framing, beam and column section sizes are driven by the physical material, its shape and building interface details, manufacture, transport and joining concepts. It is useful to contrast many modern details with the elegance of the majority of early iron and steel structures. Regular complaints from steelwork contractors about the construction difficulties leading on from designers' less than practical designs are often linked back to a focus on computer analysis and the 'minimum weight' option in software. In conceiving a steel design, the designer needs to have made conscious decisions on the buildability aspects of the design. Practical experience and buildability advice are often not readily accessible when a structural concept is developed. Adoption of some simple approaches to design/detailing and communication and observation of steelwork construction improves the chances of successful steel construction and photographs of the good and bad are provided below.

As the concept gets nearer to validation as acceptable in all aspects of design, iteration occurs in the virtuous circle of design, analysis, and detail design.

18.5.1 Design, analysis, detail design – implementation

Common practice in steel construction is for the designer/overall designer to conceive the design and the steelwork contractor to detail and manufacture the structural 'joining' details, i.e. the main structural connections. In some building forms, notably single-storey buildings there may only be a steelwork design and construct contractor responsible for overall design. In this case the contractor is responsible for overall stability. By virtue of steel material sections being high strength and dimensionally small in relation to concrete, by necessity the connections of an assembly of beams and columns are compact. Their configuration and behaviour under load directly influences analysis modelling of member assemblies.

Whatever the structural material, whether formed on site or manufactured and assembled on site, at the initial stage there is the idea, the concept. It may use bespoke and/or proprietary components but whatever the concept, 'joining' details are an intrinsic part of the concept. Some structural detail configurations will be essential for the framework to behave as predicted by mathematical models. There are usually 'key' details that drive concept decisions.

Examples are in this chapter include:

- Paired 152 UC providing flush internal wall finishes by embedding in walls (see **Figures 18.4**, **18.5**, **18.6**, **18.8** and **18.9**). The net to gross floor areas were not compromised by structural 'design development' in frame or cladding connections.
- Internal exposed soffits that dictate slab and frame construction with no down stand beams to meet M&E designer's wish for clear soffits (see **Figures 18.7(a,b)**, **18.9**, **18.10(a,b)**, and **18.11**).
- Down stand beams with 50% web material removed for M&E services (see **Figure 18.6**).

Figure 18.18 Structure-free floor depth adjacent to main vertical service riser on the right, to suit large ducts emerging into ceiling space

Figure 18.19 21+ m long roof beams

- Architectural elevations that cannot accommodate braced frames leading to a rigid frame concept.
- 21+ m long roof beams being transported to site with, lengths and internal exposed joints designed to be within economical transport limits (see **Figures 18.19** and **18.21**).
- Large or heavy members that need splicing in the fabrication works or on site depending on craneage. **Figure 18.20** shows a truss that exceeded the lifting capacity of the fabrication shop cranes so high strength friction grip (HSFG) splices were provided for site assembly. In this case, site craneage capacity was adequate to lift the site-assembled member to the third storey level.
- Heavily serviced longer floor span buildings that have localised concentrations of services that eliminate down stand beams adjacent to a primary services riser, where services congestion normally occurs.

18.5.2 Design, analysis, detail design – final design

In summary, details are major parameters affecting engineering decisions and, in the case of any structure made off site there

Figure 18.20 This truss was suspended two floors off a centre tie as the ground floor column was omitted. Requiring minimal deflection for compatibility with columns and too heavy for the fabricator to make in one piece, 2 no HSFG splices were used at each end

are issues of *manufacture*, *transport*, *erection* and *connecting* on site, a separate set of parameters to be considered. The details are refined following mathematical validation of the adjustments needed to the designer's structure. The process then iterates in a 'virtuous circle' of design, analysis and detail design.

18.6 Design parameters (cost, construction)

18.6.1 Overall cost

The typical cost build-up of a main frame based on standard rolled sections would be:

- Materials 30%
- Steelwork contractor's detailing/design 5%
- Fabrication 35%
- Priming 8%
- Delivery + erection 22%

Pricing by tonne is highly dependent on market conditions; however, there are price sensitive factors that can be assumed by the designer whatever the market conditions.

18.6.2 Material cost

A factor in material selection is the use of members built up from plate. Automated welding machinery, available in most modern fabrication shops, means built up components are widely available and specified with confidence. Fabsec and Cellform beams and even the traditional castellated beam sections are available (see **Figure 18.22**).

At first glance, there is a bewildering range of rolled and fabricated steel sections. In some construction markets, fabricated sections are the norm. Where rolled sections are available, these are likely to be the commonly used shallower sections and, if

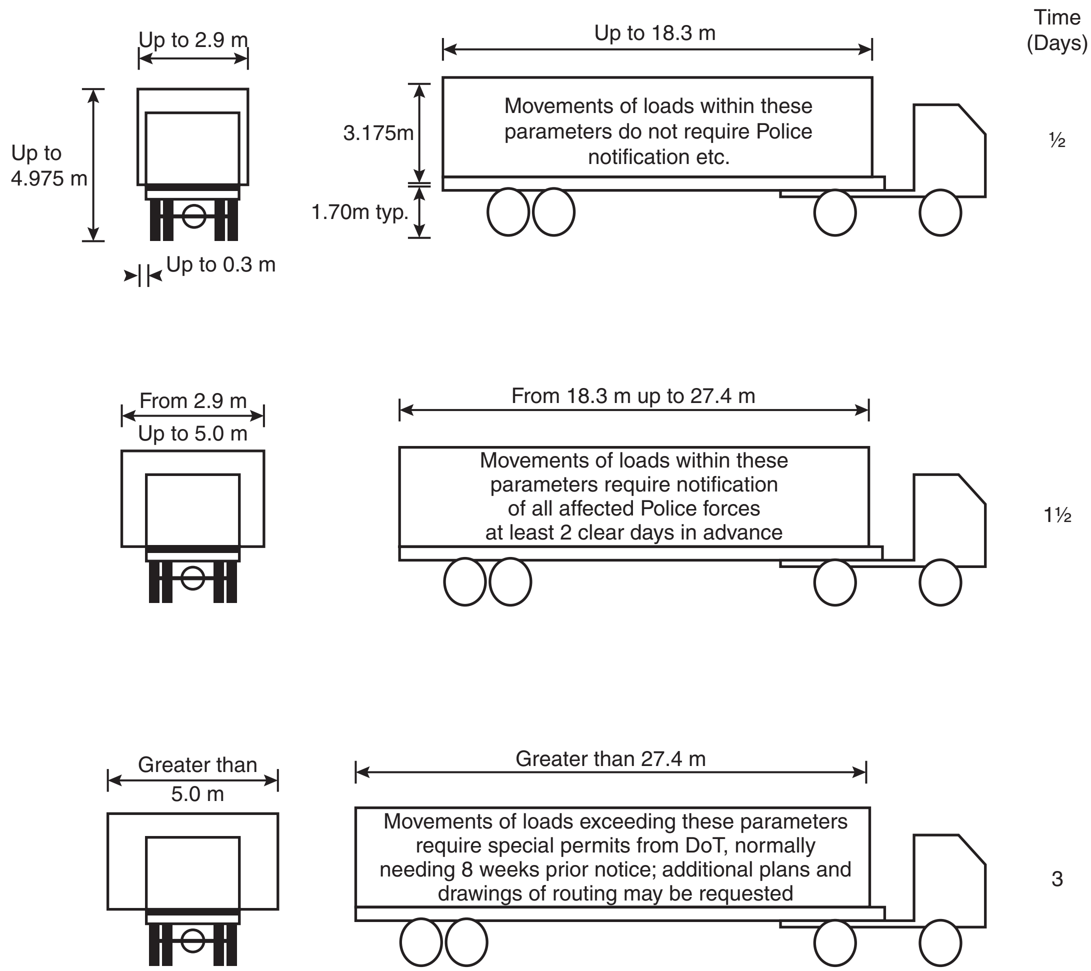

Figure 18.21 Road transport limitations (reproduced from SCI 178 (1997) with permission)

Figure 18.22 Castellated beams with re-entrant dovetail deck

imported, will be in standard lengths. The steel designer needs to be flexible in section selection while making clear in the design drawings and specifications the critical dimensional and loading constraints of the frame. In most situations in the UK, hot rolled sections from British or continental mills should be selected and fabricated sections considered in the next iteration of the design so that particular characteristics of depth/span, service/structure and weight/economy can be achieved.

For best economy rolled sections are pre-ordered to suit mill rolling dates. Sometimes steel stockholders can supply particular sections but economy will suffer. If rolled sections are not available to suit the fabrication, programme sections will need to be changed or fabricated from plate.

Costs rise for material not ordered from the rolling mill because stockholder lengths are in the range 10–12 m. For example, if 7.5 m beams are required, offcuts are scrap. Mill orders need to be in 5–10 tonne lots so the designer should limit the number of different section sizes selected for the project. The designer can review TATA's rolling dates for UK steel supply if the programme is critical.

Review of a typical three-month mill rolling programme shows that most sections are rolled three times with popular sections rolled four times a year.

Unpopular sections are rolled once a quarter. It is worth avoiding these to use heavier popular less costly sections in the same depth range. A 'minimum weight' design could be a high cost design if infrequently roll sections are specified, so it is suggested that:

- When specifying UBs avoid the narrow flange serial sizes such as: 457UB152; 356UB127; 305UB127; 305UB102; 203UB102.
- Avoid use of UB <203. These sections can be costly to connect due to small proportions. Shallow, <150 mm deep sections can be costly. A heavier common section may be more available, economic and require a thinner intumescent fire coating.
- Channels deeper than 200 mm are costly – use UBs if possible.

Avoid specifying an excessive variety of serial sizes. Maximum optimisation might be the use of three section sizes in a project: a column, a primary beam and a secondary beam (see **Figure 18.23**). Hollow sections and smaller sections are perhaps rolled once in a quarter and minimum order tonnages should be checked with the mill. Remember stiffening of open sections subject to torsion is likely to be more costly than using hollow sections.

Sometimes if loads are heavy or there are long spans, rolled sections are replaced with special fabricated sections. Before this decision, a cost effective solution for resisting a heavy load is to use twin rolled sections particularly for a transfer beam or if there is a limit on the maximum depth allowed. Where down stand floor beams are used if the services zone shares the structural zone holes may need to be cut or a Cellform section selected. For preliminary cost purposes the uplift on the rate of a basic unfabricated piece of steelwork is:

- Hollow sections +£200 ($300, €240)/tonne
- Girders fabricated from plate +£250 ($400, €300)/tonne
- Cellform beams + £200 ($300, €240)/tonne

It is worth noting that Cellform beams have limited suppliers with the potential for a cost premium. Connection positions normally need the cells filled in with welded plate and if connections are frequent use of a UB or plate girder cut with circular holes can be advantageous on cost and/or programme criteria.

18.6.3 Fabricated steelwork cost – columns

Once order of magnitude calculations determine approximate column section sizes practical framing decisions are needed such as:

- Multi-storey columns can have a section size change to suit storey loads but compare the cost justification of a splice, perhaps £150–200 ($300, €240) with the cost saved by section weight reduction. A safe initial assumption might be to change section size every three storeys.
- Consider using a minimum number of maximum column lengths consistent with transport, erection and crane capacity.
- The lowest storey column design will normally be the critical load based on buckling about the weak axis dictating the section size of, a UC. A heavy load, say from localised roof plant, can affects only a few columns consider welding plates across the toes of the ground to the first floor section, say 500 mm from the base plate up to within 500 mm of the first floor connection. This avoids any change of the column section size at the beam connection points maximising connection repetition.
- Use higher grade steel to minimise column sizes. Consider using countersunk splices to maintain a minimum standard size column casing throughout the building (see **Figures 18.24(a,b)**. This standardises column to ceiling finish junctions and maximises net lettable floor areas.
- Perimeter columns often require corrosion protection, perhaps galvanising if set within the walling thickness to minimise/avoid loss of lettable internal space from column bulkheads. Identify the

Figure 18.23 Sometimes the main framing members need only three serial sizes to be ordered in bulk by the steelwork contractor, assuming a 'minimum weight/high complexity' frame has been avoided

Figure 18.24(a) Compact steel detailing at column splice positions avoids costly change to column casings and ceiling detailing

Figure 18.24(b) Compact steel detailing at column splice positions avoids costly change to column casings and ceiling detailing

details/members requiring special anti-corrosion coatings and if galvanising is needed consider where/how non-galvanised steelwork is connected.

- Use minimum four bolt holding down (HD) arrangements for erection safety; consider glued, drilled in bolts using a template in mass concrete bases; use cast in bolts and cones for reinforced foundations. Maximise tolerance allowance in the HD bolt lengths and thread lengths. Remedial work is costly (see **Figures 18.25** and **18.26**) and if the concrete base is set too high this can require taking down the column.
- Where columns connect direct to plunge columns (see **Figure 18.27**) there are very limited options for detailing in tolerances. Normally a guide frame within a steel upper casing in the pile bore is used to place the shaft.
- In single-storey buildings, consider the need for heavier HD bolt assemblies for fire boundary conditions and overturning moments on gable and main portal columns.
- In braced bays there are likely to be non-standard HD assemblies. Check the magnitude of net uplift in lightly loaded columns and provide adequate HD tension strength/stability against overturning in the substructure.
- Thermal expansion/contraction in long frames that are restrained can exceed the lateral forces and the tension loads in column bases derived from wind loads.

18.6.4 Fabricated steelwork cost – beams

- Sometimes a 'column free' floor option is required. Doubling the secondary beam span from 7.5 m to 15 m adds approximately £30 ($50, €35)/m^2 to the structure cost.
- Sometimes a shallow floor beam option is required using deep deck or precast slabs. UC sections will require approximately double the weight of an equivalent UB for the same span/load. The secondary spanning beams in a medium rise frame with composite decking comprise approximately 40% of total tonnage. If UC sections are used with deck or slabs beams will increase 'normal' overall frame weight/cost by 30% but provide better chance of reuse at the end of building life.
- In a simple frame five basic sections can be used throughout: an internal and perimeter column section; a primary, deep/heavy beam section; a lighter secondary beam section; and for trimmers and short or lightly loaded spans optimise the use of one section size throughout. Avoid multiple section sizes and sections <150 mm.

Figure 18.25 Substructure/steel interaction detailing without generous tolerance allowance leads to costly remedial work; avoid 'economising' on HD bolt length

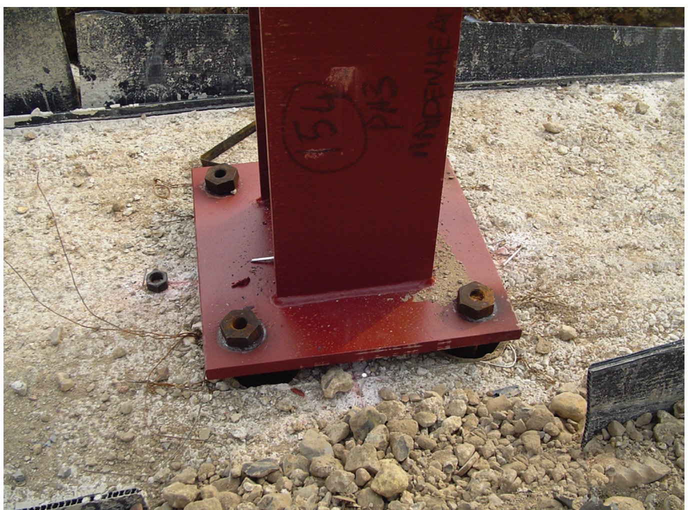

Figure 18.26 An unacceptable site modification to address HD problems

18.6.5 Fabrication costs – general

- Simple frame connections can be defined in accordance with the three types described in *Joints in Steel Construction: Simple Connections* (SCI and BCSA, 2009). Sometimes full depth end plates are advisable on primary beam grid connections to columns to facilitate temporary stability with fin plates used on secondary to primary beam connections.
- Avoid specification of multiple hole diameters so computer-controlled machine drill bits do not need changing. Consider the use of fully threaded bolts so that bolt lengths can be standardised and in a single diameter or a limited number of diameters. For example, the M20 × 60 mm long grade 8.8 fully threaded bolt will be suitable for 90% of the connections in a typical multi-storey frame.

Figure 18.27 Exemplary substructure/steel interaction tolerance control on the verticality of steel plunge pile heads (painted red, note web hole for lifting pin) connected to the superstructure

- Web holes required for the coordination of structure and services need checking to avoid web stiffening wherever possible. If stiffening is on many secondary beam spans it can be cost effective to change to a heavier section with a thicker web.
- Avoid joining the same depth beam sections so that double notches are avoided. If double notches cannot be avoided check this does not reduce web shear strength below the load applied. Alternatively square cut the beam end and design a longer fin plate for the increased eccentricity of the bolts.
- Splices in heavily loaded trusses and transfer girders, where deflection control is critical, may need relatively costly HSFG assemblies.

18.6.6 Building detail interfaces

The steel frame is one of the first parts of the building erected on site often before follow on detail development. Where possible, provision for interfacing with follow-on trades and a plan is needed to manage information and site modification if incomplete information has been provided to the steelwork contractor.

- For steel to timber connections, drill 20 mm staggered holes at 500 mm centres in top flanges and/or holes at 500 mm centres in webs to allow bolting of timber plates to steel (see **Figure 18.28**).

Figure 18.28 Web holes predrilled for through bolting of timber plates to fit timber joists at the same level as the steel

Figure 18.29 Concrete drilled in fixing with slotted steel fin, three steel-to-steel connections with a single steel to concrete connection to simplify tolerance design

- For steel to concrete connections, allow for tolerances and design for the bounds of eccentricity (see **Figure 18.29**).
- Control level and slopes of composite floor decking by beam pre-cambers to address dead load deflection. Change the decking to one with higher stiffness if deflections of 15–20 mm due to ponding of wet concrete are a concern. Consider and control/specify concreting method/day joints, for example an exposed soffit.
- Cracking over supports in composite slab construction is common and accepted for offices with raised floors. In cases where brittle finishes are applied directly to the slab use floating reinforced screeds. In situations where vinyl flooring finishes are used ensure the finish has the required crack bridging properties.
- In tall, 12+ metre high single-storey frames where internal shelving/racking is close to columns, eaves sway, when combined with construction tolerances can cause clashes. This is solved by increasing the building size or stiffer columns.
- Masonry clad single-storey buildings are less tolerant of eaves sway than metal clad buildings. Adjust column size to address the limits allowed for masonry panel deformation. Refer to *Brick Cladding to Steel Framed Buildings: Commentary* (BDA, 1986).
- Control/limit the imposed load deflection of perimeter floor beams to span/500 for beams supporting masonry and brittle cladding.
- Base plates of internal column details adjacent to or supported on lift pit walls cause local substructure detailing problems and pits may not be constructed early in the programme. Consider supporting the column base plate on the base of pits with pit walls concreted after steel erection.
- Single-storey buildings often have ground floor slabs that are not restrained. Top of foundations need adequate allowance for: depth for slab, sand slip layer, sub-base, grout and plate thickness. HD bolts need to be accommodated within the sub-base, possibly resisting horizontal loads.
- Base plates on perimeter columns should integrate with perimeter walling/damproofing details. Consider the use of offset base plates to facilitate walls detailed to run past columns.
- Steel material is rolled within a range of rolling tolerances. Fabrication and erection are achieved within certain tolerances. Cladding zones need to be 'loose fit'. Structural interface tolerance summaries advised to the architect for incorporation in the cladding specifications need to allow for all the tolerances likely to occur.

18.6.7 Visually exposed steelwork

The designer would normally clarify aesthetic details requirements. In addition to special welding and plate profiling permanent deformation, due to dead load actions added to tolerance effects needs considering to avoid unsightly steelwork (see **Figures 18.30** and **18.31**).

- Web stiffeners in open sections cut back 10–15 mm from flange toes preserve the visual effect of straight flanges and the fillet weld can be kept back from the toe edges to avoid bulky weld groups that are apparent in **Figure 18.34** and **Figure 18.35**.

Figure 18.30 Aesthetic detailing, stiffeners set back, column head detail accommodates restraint tie in an industrial building

Figure 18.31 Uncontrolled fabrication marks in finished steelwork

Figure 18.32 Plan on welded corner junction of 200 × 200 SHS, expensive mitred corner detail avoided avoiding problems connecting the 2-bolt post connection from below

Figure 18.33 Near perfect splice fit achieved because the same piece of SHS has been used for both sides of the splice; note longitudinal seam in this section, good for accuracy but might compromise aesthetics if exposed

- When joining open sections, if possible select beam widths less than the column flange widths while avoiding use of the infrequently rolled narrow sections. When joining beams to column webs select beam widths less than the distance between the root radii.
- With tube work remember there are limits on the lengths available. The fact that they are relatively infrequently rolled could require sections to be supplied by a stockholder so waste could be high.
- If cold formed hollow sections are used, while the surface will be smoother and section dimensional tolerance better than hot rolled, there is a longitudinal welded seam that may not be aesthetically acceptable.
- When using hollow sections in trusses try to avoid the special profile cuts needed at the junctions of three or more members. Mix

Figure 18.34 Exposed steelwork can become a maintenance problem

Figure 18.35 Exposed steelwork can become a maintenance problem as this 10-year-old example shows; galvanising should be considered the norm for any external steelwork

circular with square, rectangular or open sections to simplify joint welding. Note open sections will appear lighter and less visually obtrusive than hollow sections due to shadow effects.

- Use countersunk flange splice plates for joining open sections. Countersinking requires the use of two drill sizes so ensure the requirement is stated in the pricing information.
- Splices, in tubular work may need to be ‘matched’ by selection of the same tube length each side of the ‘cut’. See **Figure 18.33** where a structural hollow section (SHS) is splice bolted in the same member and saw cut so that distortion tolerance common on tubular cross-sections does not prevent fit up. Through wall bolting needs spanner access that can be provided by large cut holes in lightly loaded sections (see **Figure 18.32**). In this case, the holes also allow insulation fill of the section to address cold bridging.
- Dome headed nuts can be used in combination with fully threaded studs but must be specified at the time of tender enquiry for aesthetic/exposed tubework.
- In some situations the straightness of standard rolled steel sections can be inadequate/visible. Re-rolling to closer tolerances than the standard requirements of the execution class in BS EN 1090-2:2008 *Execution of Steel and Aluminium Structures* (BSI, 2008) is possible but costly, so consider using aluminium.
- Permanent vertical deflections/sag in members due to dead loads can be unsightly in exposed steelwork. A vertical deflection/sag of span divided by 250–300 will be visible. It is also worth noting that a 9 m long 305UB could be bent up to 9 mm at mid span and remains within normal delivery tolerance to the steelwork contractor, potentially additional to the dead load sag. Beams and trusses should be specified with a camber equal to the calculated dead load deflection plus another 20–100 mm, dependent on the member length, to avoid visible sag.
- Slimflor systems sometimes require special connections minimising structure thickness. **Figure 18.36** shows an example of a 9 m continuous twin beam supporting 7.5 m span precast slabs. Note the bottom flange plate is shop welded to one side of the splice, a downhand site weld completes the partial strength connection.
- Composite decking deflections are visible being approximately span over 200 under wet concrete loading. This is tolerated in normal construction but might not be acceptable if the slab soffit is visible. Consider a change to a deeper deck or use precast flooring if a slab soffit is to be exposed.
- Exposed slab soffits are currently popular and a steel frame option with precast concrete is worth considering. Beam sections such as the ASB or a UC with welded soffit plate are common or in the case of a two bay wide floor plate, precast planks can be rested directly on UC beam bottom flanges. Precast soffit void drain holes are visible in hollow core units. Better looking but more costly Ommia precast slabs can be used but temporary propping would be needed to achieve the same spans.

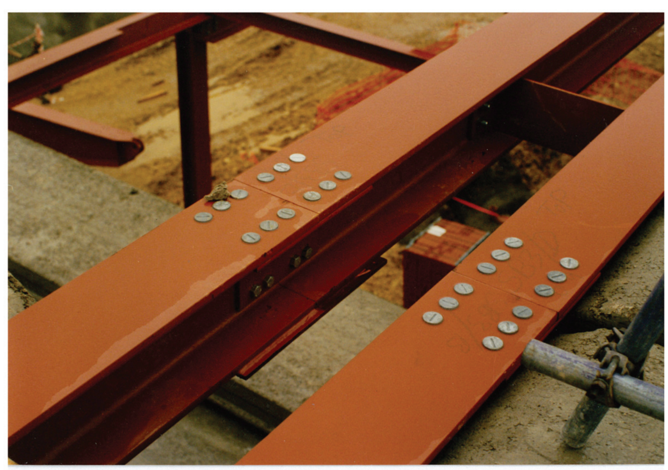

Figure 18.36 Slimfloor twin continuous beam and precast

18.6.8 Pricing documentation

Unsuccessful steelwork construction is characterised by a lack of attention to the detail of the tender enquiry. Section 1 of the BCSA's *National Structural Steelwork Specification for Building Construction* (BCSA, 2007) provides an information checklist. In addition consider:

- Any secondary structures that might need to be provided, i.e. what is not drawn but assumed to be included in steelwork supply.
- Composite decking manufacturer's edge details parallel to the direction of span may show no support. Some cold formed deck edges are torsionally stiff enough to contain wet concrete but longer span, >3 m, should be checked. Invariably there are other building fabric connections to be made that require trimmer beams.
- When the designer does not provide decking drawings, interface management suffers. If the designer relies on the steelwork subcontractor's decking supplier to provide the decking layout, coordination of openings needs managing. Both slab mesh reinforcement and the loose reinforcing bars potentially needed to address longitudinal splitting of shear stud to concrete will need defining by the designer.
- Numbers/spacing of through deck shear studs for composite beams and temporary construction propping if required.
- In single-storey buildings purlin/rail bracketry, spacing and depths impact on substructure and interface details. If a parapet upstand is used to disguise a pitched roof it needs adequate height at the ridge flashing. Purlin depth affects the parapet height round the whole building. Rail spacing/depth affects the steelwork to ground floor edge detail. If a designer does not define the cladding zone, it will not be possible to set out the ground floor edge. This could cause programme difficulties unless the steelwork supply/detailing is confirmed early in the programme.
- Roof bracing can obstruct craneage access to floors for installation of precast planks/staircases. Consider designing the bracing away from these areas and/or combine precast supply with the steel frame.
- Vertical and horizontal restraint cladding connections can require costly and disruptive site modification. Consider the details/connections for main frame to secondary framing. Sometimes self-spanning cladding delivered in large formats are preferred to alternative small format cladding that requires secondary framing support of multiple pieces. If large format cladding is used, it needs connecting robustly to rigid positions, i.e. column grids, in the frame and heavier fabrication identified before pricing (see **Figure 18.37**).
- Storey height precast cladding units supported off the ground avoid the need for the frame to support heavy edge loading. See **Figure 18.38** for an example where the facade was supported off the minus one level restrained by a steel frame with vertical slotted holes. **Figures 18.39** and **18.40** show the erection-levelling stub in the back of the precast calling panel supported off the frame, an example of the steel frame designer coordinating the erection needs of the cladding solution.
- The support on roof and edge details needed for cleaning and maintenance access machinery.

Figure 18.37 This heavy beam end connection provided support to both horizontal and vertical precast concrete cladding, some of the units weighing 8 tonnes, spanning 7.5 m

Figure 18.38 Storey height precast cladding units supported off the minus one level restrained by steel frame; avoids need for frame to support heavy edge loading

- The difference in plant room floor level and construction with the roof usually requires steel level changes. This is best solved by using two adjacent beams at different levels or one sloping. Attempts to use one beam with fabricated bracketry are usually costly.
- There is a practical limitation on the accuracy in achieving pre-cambers. Avoid specifying <20 mm camber.
- The need for service holes in down stand beam solutions and unknown setting out at tender stage can be a challenge in heavily serviced buildings. Effort is needed to determine shapes and maximum sizes likely from the service designer. The need for web stiffening should be identified at tender. **Figure 18.41** is unusual in that the services and structural designers were able to coordinate at the appropriate time before fabrication, crucial where multiple holes were required in a shallow beam. **Figure 18.42** shows the results of no coordination.

Figure 18.39 Rear of storey height precast cladding before removal of the erection shims; see Figure 18.40

Figure 18.40 Rear of precast cladding central levelling/lining stub on the steel frame, shims removed, after cladding bolted through vertical slotted holes in the column (horizontal restraint only)

- Full depth end plate connections should be provided in 'simple' construction where beams are likely to be unsymmetrically loaded at construction stage, for example, with precast slabs.
- If the steelwork contractor is expected to liaise with other contractors, make this clear in the tender documents. This is particularly important for precast stairs and slabs; decking if by others; service hole penetrations; cladding and secondary support.
- Coating requirements could include: no paint for internal dry conditions; galvanising with 140 microns for external exposed; 75–80 micron zinc phosphate or similar primer; primers specified specifically for finish coating by others need to be compatible with intumescents. Check for pale finish coats that will require pale primers.
- The allowance in terms of tonnage or cost needs to be identified for items such as: connections 5–8% of the whole tonnage; design development; secondary steelwork not identified on drawings; trimming for service holes.

18.6.9 Note on design codes

It is likely the reader has been carrying out designs in accordance with Eurocodes at university or. If in practice the reader will be used to the British Standards while at the same time being aware of the debate over the timing of formal adoption of the Eurocodes in the UK Building Regulations in 2013. International steelwork designers are likely to use the codes produced by the American Institute of Steel Construction (AISC) and the American Iron and Steel Institute (AISI) with seismic designs to the International Building Code (IBC). Steel grades tend to be lower than those used in the UK and the resistance of the frame to seismic effects controls joint detailing and ductility provisions.

In the UK introduction of BS 5950 in 1986 involved replacement of BS 449, a permissible stress code, with a limit state code. Industry took time to align itself with BS 5950 and

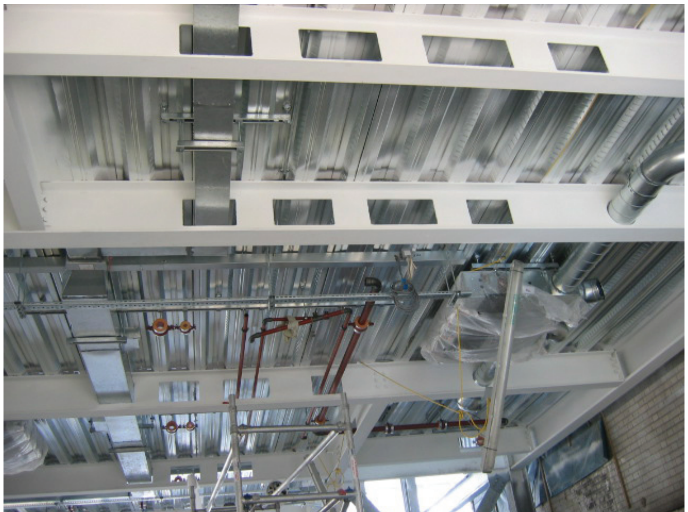

Figure 18.41 Multiple holes coordinated, crucial in a shallow beam solution

Figure 18.42 This example of site burnt out web holes shows what can happen if coordination is not clearly communicated, leading in this case to costly remedial work

some designers still design satisfactorily using B S449. Points to note are:

- Use of a 'Standard', be it new or old, does not confer legal immunity.
- The designer's work is judged on being reasonably competent, while
- Not wishing to support the use of permissible stress methods, the use of permissible stress codes will be adequate for some situations.
- Preliminary concept design work might well be based on broad principles based on simple codes devised at a time of limited computer use.
- Whatever the level of sophistication of the computer analysis, it is only a model. Steel's ductility is a great 'forgiver' of minor error between the imagined model and real behaviour.
- Basic principles governing the behaviour of steel sections such as buckling, stability, elastic and plastic characteristics of sections and assemblies of members are constant whatever code is used.
- 'Failure' of a design to meet a limit state, be it serviceability or ultimate will occur when the designer's analysis model does not match real behaviour or when the 'actions' have not been assessed adequately.

In the UK, Eurocode terminology should be used to ease communication of design. Chief differences between the Eurocode and traditional terminology are:

- Actions = loads, imposed displacements, thermal strains
- Effects = bedding movements, axial forces, shear, etc.
- Resistance = capacity of an element to resist the 'Effects'
- Verification = check
- Execution = construction, fabrication, erection

Mixing of code partial safety factors was not an uncommon error when the limit state codes CP 110 and BS 5950 were first introduced. Superstructure designers using factored loads are used to providing unfactored loads for substructure.

It should be noted that a site investigation in accordance with Eurocode recommendations is more comprehensive and will be more costly than current practice. Limit state design approaches in the UK for geotechnical work are less well developed than structural design practice and it will take some time for the two to converge. Mixing of BS and Eurocodes will hence be the norm for the near future.

Both designers and steelwork contractors are active in use building detail design. There will be situations where 'actions' and 'effects' derived from BS EN1990/1991 will need to be used for elements in a frame, and connection designed based on moments, shears and axial loads as per BS 5950. If the 'resistance' of elements is determined from BS EN1990/1991, it is possible for connections to be designed to satisfy BS 5950.

One can imagine that at an interface already quite involved in terms of design responsibility, information from the non-structural team members, and main contractor programme demands, now pose an added risk in the communication of the designer's design intent.

The Eurocodes are not material specific with regard to analysis, but set out 'Principles' and 'Application Rules'. The National Annexes provide alternative factors where the Eurocode permits a change and if a national committee requires. Design methods can be advised and reference made to non-conflicting complementary information (NCCI).

Guides on the Eurocode for steelwork design include the *Manual for the Design of Building Structures to Eurocode 1 and Basis of Structural Design* (IStructE, 2010). Other references such as the 'UK Eurocodes' series of SCI publications dated 2009, including Brettle (2009), which provides an introduction to the Eurocodes with reference to steel building design, can be used for frame member designs to Eurocodes.

18.7 Preliminary structural steelwork arrangements

A designer starts with architectural plans and elevations, a building concept from the architect. There is a sequential procedure for devising a structure taking into account:

- column grid study;
- stability provision, expansion joint philosophy;
- roof construction, shape, parapet detail and elevation treatment;
- floor construction study;
- interfaces with architectural and M&E requirements;
- element studies on floor and wall thickness;
- preliminary beam and column sizing;
- analysis and design iterations.

18.7.1 Column grid study

The overall typical footprint dimension is assessed and columns positioned in clear space and in perimeters. At this stage, the cladding/walling type and thickness should be considered and an initial setting out of the perimeter column determined. Elevation treatment, say with windows, a curtain walling stick system or precast panels will influence grid spacing. Internal columns may be on the same grids as the perimeter but variants can be devised that provide perimeter columns more closely spaced than internal.

Low buildings with lightweight roofing may have a clear span top storey that is cost effective . In this case so the internal intermediate column would be one storey less than the perimeter columns

18.7.2 Stability provision, expansion joint philosophy

Braced or rigid framing needs an early decision. Bracing is the norm for economic construction but its location needs agreement with the client and team. The plan shape of the building will also influence the extent of bracing. Often restrictions on the positions where bracing can be located lead to unbalanced stability with the building potentially twisting in plan under lateral loading. Measures to counter this are adjusting the orientation of the columns if an open I-section is used so that the stiffer X–X axis of the column helps to reduce the lateral deflection of frames remote from stiff bracing, or the introduction of rigid joints in some of the frames can be considered.

If rigid framing is adopted, beam to column junctions will be costly to fabricate. Typically, the column size/weight will double if rigid framing is chosen for a modest height frame up to say four storeys. Lateral deflections will need early consideration with respect to tolerance of cladding to deformation. Sometimes designers make the mistake of selecting a rigid frame with slender heavy columns that, even with a fully rigid beam to column joint, have a low stiffness and sway excessively even to the extent of necessitating p-delta calculations on the columns. If this is happening, the concept needs challenging.

18.7.3 Roof construction, shape, parapet detail and elevation treatment

Flat or sloping roofs need a preliminary assessment of possible plant weights and the likely method of inducing falls, in the roof build-up or by sloping the structure. Beware of screeds laid to falls that can become thick once the drainage points are located, may be limited in number. Dimensional requirements on flashings and edge up stands that accommodate roof falls can use up overall building height allowances and impact on the structural zones allowances for the roof and floor structures, so a limited amount of early detail iteration is needed.

18.7.4 Floor construction study

Floor thickness studies determine the range of options for the structural zone and if it is to be shared with the services or if there are special requirements for exposed soffits. This is an increasingly important requirement as natural ventilation and mixed mode ventilation systems become more popular

Table 18.1 shows the variants that may be considered. A 'column free' alternative is included – often considered at an early stage in the design to maximise flexibility of the space. The client, depending on building use, sometimes agrees dimensional constraints on overall building/floor height accepting the cost penalty of long span construction.

	7.5 × 9 m grid	7.5 × 15 m grid
1	Steel composite beams and composite slab	Cellular/Plate girder composite beams and composite slab, max depth to be reviewed with architectural and services requirements
2	Steel frame and non-composite precast concrete floor	Down stand steel UBs with composite slab with discrete web holes
3	ASB or fabricated Slimflor beams with either deep deck or precast slabs and down stand edge beams	Conventional steel UBs with composite slab with discrete holes but restricted to depth of option 1

Table 18.1 Variants that may be considered for floor construction

18.7.5 Interfaces with architectural and M&E requirements

Interpretation of the architectural characteristics requires iteration between detail construction of the building fabric and likely thicknesses of finishes and fixings that drive the sizes and location for the structure. This most critical stage in the designer's interpretation of the building and the appropriate structure requires peer review and coordination with the team.

18.7.6 Element studies on floor and wall thickness

Simple member designs determine the order of magnitude of structure integrated with the output from the review of the architectural and M&E requirements.

18.7.7 Preliminary beam and column sizing

Figures 18.43 and **18.44** summarise the options for a simple two-storey 18 m wide building. This work would allow the client and design team to agree the frame topology that is subsequently modelled in detail, probably in 3D software.

18.8 Challenges and opportunities

So how do we draw on the design and detailing skills of the designer, and the contractor making the structure? The steelwork industry has a tradition founded in craft-based workshop skills. The rigour imposed by modern methods of factory production has, by necessity, led to close integration of design and construction techniques.

Integrated steel design and manufacture is comprehensively reviewed in the 'Computer integrated manufacture of steel' (CIMSTEEL) explained in the two publications *Design for Manufacture* and *Design for Construction* (SCI, 1995 and 1997). This initiative has not had the publicity it deserves and the reader should consult these references.

The concrete industry has made significant advances over the last few years in terms of rationalising site construction, design aids, pre-fabricated reinforcement assemblies and advances in off-site production of components. There is a convergence where the designer/consultant integrates the work of specialists in concrete structures in the same way as has been the case in steel structures for some time. Just like steelwork, success will depend on concrete details developed that satisfy the parameters of design and practical construction.

18.8.1 Hybrid construction

More complex buildings using mixed structural materials is common as designers respond to the challenge of integrating structure with building services and architectural challenges of exposed structures and natural ventilation. Demands for rapid construction on site and continued pressure on price and cost of labour acts as a further impetus to use off-site manufacturing methods.

Hybrid construction (see **Figure 18.45**) involves the combination of site and off-site manufactured steel and/or concrete members. This form of construction seems likely to make an increasing impact on the UK. Hybrids exploit the most favourable structural characteristics in the materials.

In buildings that incorporate different structural materials, there are key architectural and structural details that drive the structural design.

A mix of architectural and building service parameters will tend to dictate structural positions and shapes. The structural 'joining' details will generally incorporate high strength material to facilitate factory manufacture, delivery and erection. There is a large body of research and knowledge in the design and detailing of conventional steelwork and, for example, in the UK the steelwork industry remains at the forefront in the off-site production and site assembly of structures.

Challenges of interface management and by implication design of details will become crucial for designers and contractors. Expertise in hybrid construction will be important for designers.

18.8.2 Reuse/recycle

Sustainable design with particular regard to future dismantling and reuse of structural materials will introduce additional design considerations in all structure types.

The realities of the market for second-hand construction materials suggest it is challenging to successfully dismantle and sell on any complete frame except the most straightforward single-storey buildings often finding a second use in agriculture.

In the case of multi-storey office-type buildings steel framing with precast floors is simpler to dismantle and reuse. In an assessment of a building framed in composite decking, composite steel secondary beams and non-composite primary beams buyers could be found for only 107 tonnes of the 670 tonnes of steel in two buildings. A price of £260 ($400, €300) per tonne (2004 prices) for the 107 tonnes was offered but extracting that tonnage was not financially viable. A demolition contractor advised at the time that since prices for scrap were high it was not worth their while to painstakingly dismantle the frames, bolt by bolt. Other issues emerged such as:

- A frame with composite secondary and primary beams with through-deck welded studs will not easily be separated from the composite slab; use of a non-composite precast might be more attractive though not necessarily on cost grounds.
- Column reuse would be straightforward.
- 75 microns of paint were specified but there were 400 microns in places. 400 microns would significantly impede reworking/welding of steel sections retrieved from the building for reuse.
- Galvanised sections were a problem for any reworking/welding of steel sections retrieved from the building for reuse.
- Board cladding for fire protection, used in the past, might be a regressive step but does not pose the same reusability problems of thin cost effective intumescent coatings which are the current norm.

Demolition contractors were offering between £50 ($80, €60) and £80 ($125, €95) per tonne for scrap making demolition a cost-neutral exercise, as the cost to demolish would be around the same as the price given for the steel.

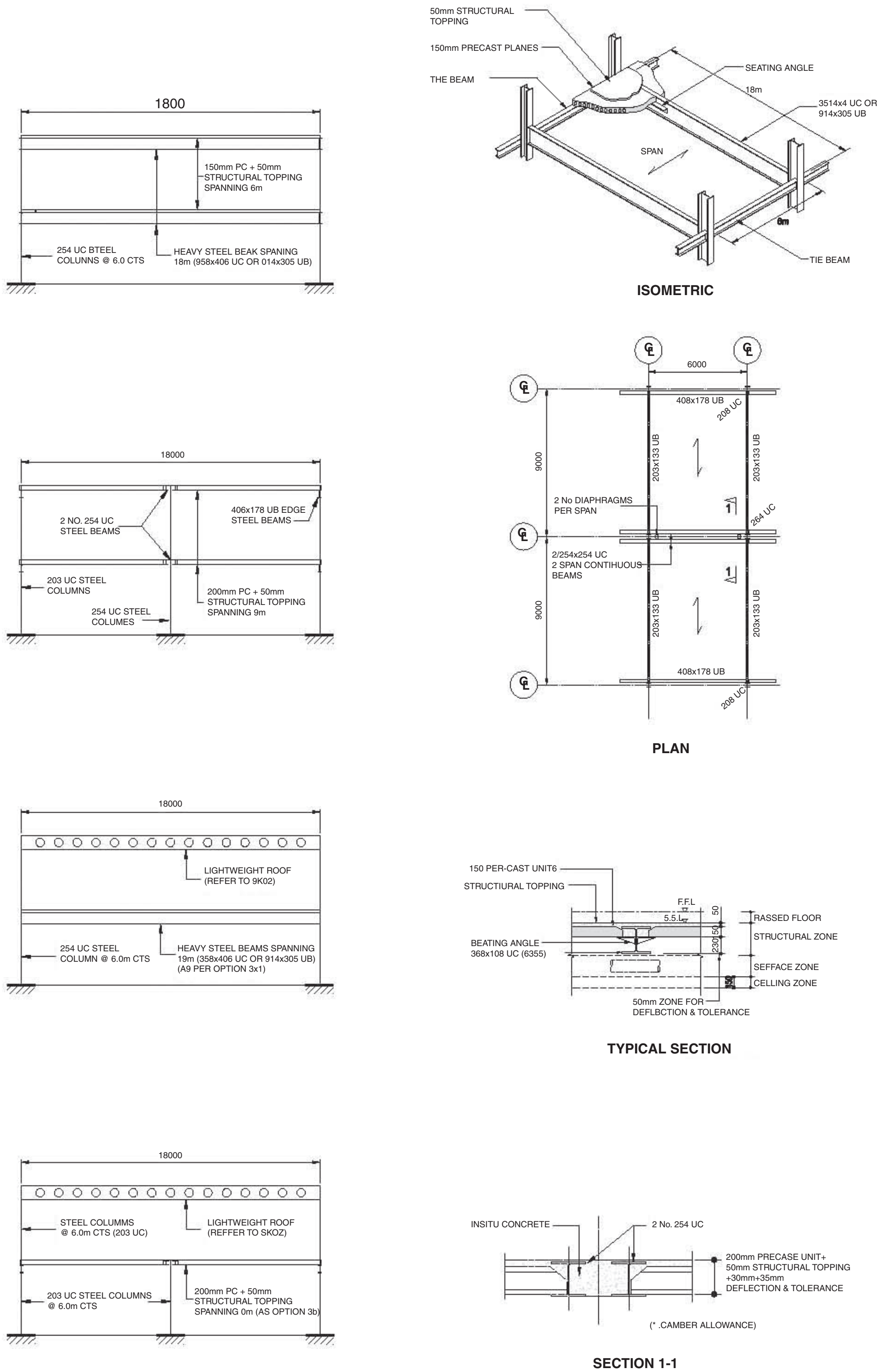

Figure 18.43 Column and beam span studies – precast

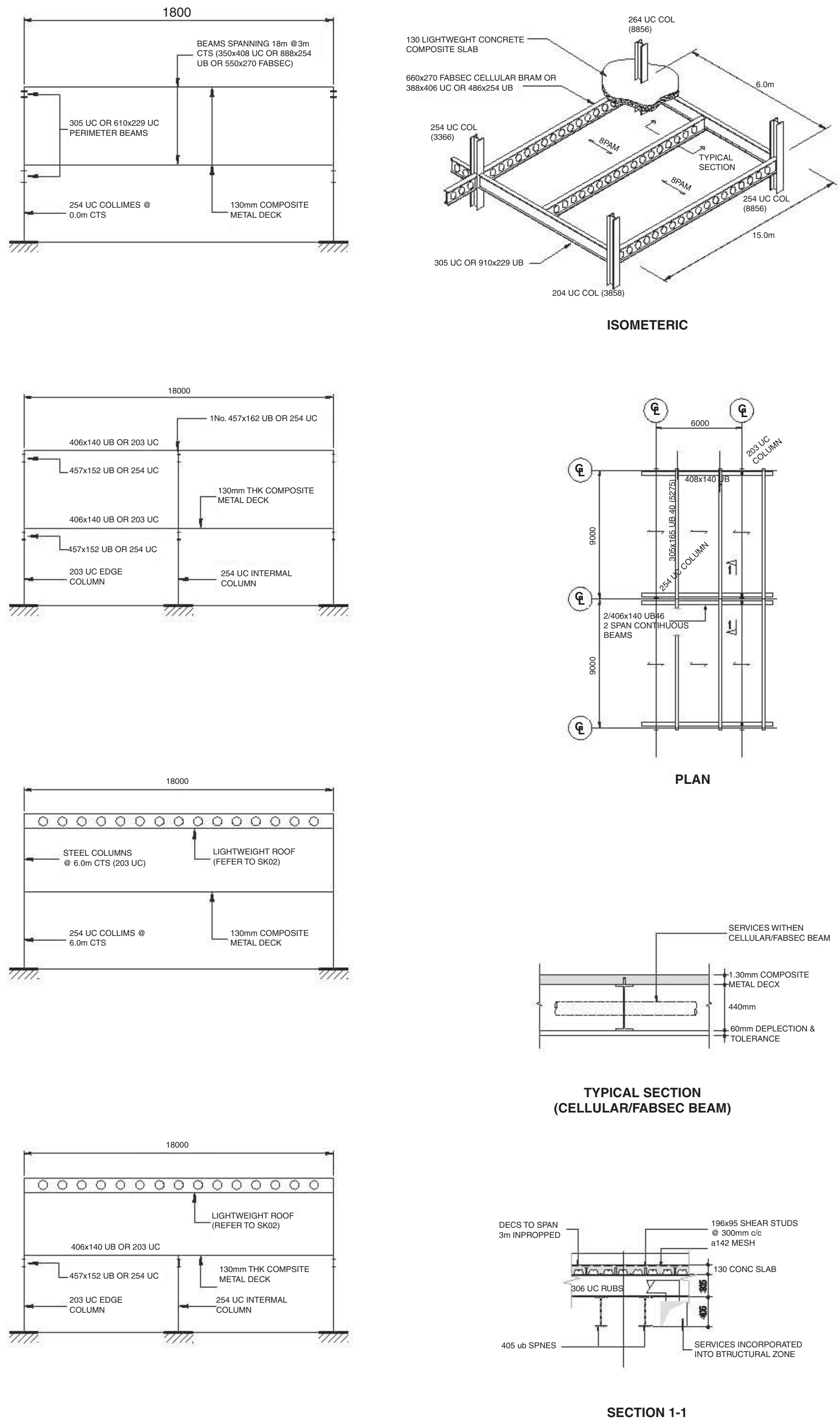

Figure 18.44 Column and beam span studies – composite

Figure 18.45 Hybrid structure in steel and timber

Figure 18.46 68 King William Street after refurbishment

Figure 18.47 Concrete encasement broken out at connection

In summary while a design warrant for reuse, subject to a check, should be relatively simple to arrange, perhaps through the original designer, when a building frame accounts for less than 5% of the overall cost of a building, a developer or contractor is unlikely to take on the perceived 'risk' of using second-hand materials.

18.9 Case study of the reconstruction of a 1920s steel framed office building

18.9.1 Background

The former Lloyds Bank/Guardian Assurance Company building at 68 King William Street was a typical urban regeneration project in the City of London. The building was directly over the Docklands Light Railway tunnel at its junction with Monument Underground station and in a conservation area adjacent to the Grade I listed St Clements church (see **Figure 18.46**).

The original building was erected between 1920 and 1922. The steel frame provided nine floors plus two basement levels. Each floor plate was approximately 1000^2.

Two different contractors had originally been involved. Clay pot and concrete rib floor slabs were in the eastern half of the building, the western half used steel filler joists and concrete in-filled triangular clay pot slabs. Steel beams and columns were concrete encased (see **Figure 18.47**), and either single or double UB sections with riveted plating to the flanges. These were likely to have been shop riveted, brought to site and in the case of columns, spliced with on-site riveting. End connections were typically bearing angle cleats top and bottom.

The facades were constructed in Portland stone with brick backing or glazed brickwork in both cases built monolithically around the steel frame and concrete floor slabs. There were numerous areas of slip bricks in locations with unprotected steel 30–40 mm from the external face. The monolithic facades were retained and the internal shear walls provided stability (see **Figure 18.48**).

18.9.2 Refurbishment aim

A conversion to a mixed-use building was required with five lower retail floors and five office floors above. The five upper floors were demolished and rebuilt using the minimum possible structural depth to increase clear ceiling heights, while retaining the facade and rationalising the office column grid by the introduction of a transfer structure at level four. Below this level, the original structure was retained (see **Figure 18.49**).

18.9.3 Investigations

- A historical information search yielded the original details for the eastern half of the building constructed by Trollope and Coll.
- Radar testing located existing foundation pads, trial pits and boreholes.
- Soils analysis was carried out for heave calculations due to unloading during deconstruction.
- Metallurgical and tensile tests of existing steel from coupon samples taken from existing beams and columns were carried out.
- Intrusive investigation of around 60 No. concrete cores was implemented.
- Load testing of existing suspended floor slabs was done.
- There was a precise topographical survey of the building.
- Compression testing of the original party wall brickwork to confirm retention for building stability.
- A condition survey of the tunnel beneath the building was carried out in conjunction with movement monitoring of the building during demolition and rebuild.

The existing steelwork had average yield strength of 285 N/mm^2, equivalent to grade 40–43 steel. The average carbon content was 0.2%, varying from 0.06 to 0.35%. New welds to existing steel required checks for signs of cracking.

Some original defects were apparent in the existing steel sections: hairline cracks/surface lines caused by segregation of impurities formed during the rolling of these sections. Strengthening details were designed to bridge across such defects.

18.9.4 Assessment

At the time of the original design, the London Building Act (1909) required the floors of 'counting houses' designed for an imposed load of 100 lbs/sq ft (4.8 kN/m^2). New loads would be imposed load (4+1) kN/m^2 and tenant finishes (1.75 kN/m^2).

- The retained floor slabs were justified by back analysis and load tested to determine a safety margin. Justification of the existing slabs was straightforward due to the standardisation of the reinforcement in the concrete ribs.
- Analysis of the beam and supporting riveted angle cleat connections retained below fourth floor meant that most of the connections had to be back analysed, using the information from the strength tests.
- Back analysis initially used rivet strengths and steel stresses based on the BCSA's *Historical Structural Steelwork Handbook* (BCSA, 1989).

Figure 18.48 Temporary works for the retained facade

Figure 18.49 A typical connection of the transfer beams to original columns

The diameter and number of rivets dictated the capacity of connections. The assessment of diameter was not straightforward as head sizes were variable. Secondly, the capacity of the connection varies considerably in back analysis. For example, if 4.3 rivets were originally calculated 6 might be provided for symmetry; if 5.9 rivets were calculated 6 would also be provided. Consequently, back analysis derived a wide range of potential capacities. Small components such as rivets implied that estimated sizes could dramatically change estimated design capacity. Connections were much more variable than beams requiring more comprehensive design checks and sampling.

Assessment of strength of riveted compound columns was affected by the estimation of rivet sizing limiting column capacity. The steel test results and recommendations for material factors in the SCI's *Appraisal of Existing Iron and Steel Structures* (SCI, 1997) were used to up rate estimated capacity. Columns were finally design checked to BS 449 using 165 N/mm^2 allowable stress supported by the material test results. In summary, the assessment increased in complexity with increased knowledge of structure.

18.9.5 New design

The congested city centre location meant that only one crane with 6 t lifting capacity could be accommodated through the new atrium in the centre of the building. This was a major constraint.

At feasibility stage, office floor thicknesses were assumed to comprise 150 mm o/a raised floor, 750 mm zone for structure and services and a 150 mm ceiling lighting zone. The most efficient structural floor system for the building was a Slimflor system (see **Figure 18.50**). 203 UC composite beams were used in the same depth as PMF 225 deep profile decking and a 305 mm lightweight concrete slab.

This system provided a notionally flat soffit and an overall structural depth of 305 mm for ease of services distribution in the 400 mm nominal zone provided below the structure. However, some of the transfer beams at fourth floor and roof levels were designed as 630 mm deep plate girders with web openings for services. The deep decking spanned up to 5.8 m unpropped and the lightweight concrete slab helped reduce loads on existing columns being lighter than the existing slabs by some 10–20%. In some areas, twin slimflor beams were used for spans of 11 m (see **Figure 18.51**).

Connections of new to existing steelwork below the fourth floor were designed on the basis that the original steel angle cleat connections were replicated to ensure column moments were no higher than the original design (see **Figure 18.52**). For ease of new beam installation, a detachable stub of equal section size was connected to one end of each beam, bearing onto the newly installed angle cleat providing invaluable adaptability for the steel erection while working in congested locations.

18.9.6 Cladding/facades

The existing Portland stone and glazed brick facades were removed down to the sixth floor and rebuilt to match the higher floor levels with new courses of stone or brickwork inserted at various levels. Reconstruction of the stone facade was carried out in the same monolithic construction as the original. This led to complex detailing of damp proofing and insulation details. To ensure that the facade remained self-supporting and that no additional load was carried by the existing steel structure, the edge beams and concrete slab were dry packed into the stonework. Gaps were left in the dry packing at 1 m intervals to allow for drainage, and insulation installed along the top of slab and soffit to avoid cold bridging.

The retained areas of existing steel embedded in the stone front facade were in some cases badly corroded. In accessible areas, stone was removed locally, the steelwork cleaned, coated in a bituminous paint and stonework replaced. Where embedded facade steelwork was badly corroded cathodic protection (CP) was considered to protect embedded steel. However, it required replacement every 20 years involving significant disruption to tenants. As the amount and rate of corrosion were small, and the capital cost and maintenance of the CP system high, it was

Figure 18.50 The Slimflor system used for the new floors above 4th floor

Figure 18.51 Twin Slimflor beams spanning 11 m

Figure 18.52 Replication of the original angle cleat connections with a stub to aid erection

not adopted. Maintenance of elevations would therefore be required over the 60-year design life of the structure.

18.10 Conclusion

This chapter outlines an approach to steel design that encourages study of buildings and construction details that use steel structure so that an appreciation of the practicalities of construction in steel can be developed. Steelwork structure relies on its compatibility with building fabric details to be successful. The steelwork designer cannot ignore these details.

Although not providing a comprehensive guide to steel design, this chapter summarises the key parameters considered before initial steel design should be attempted. The section dealing with preliminary design is intentionally close to the end of the chapter. There is a considerable body of published material and the reader is encouraged to continue to study.

Looking to the future of steelwork construction and its sustainability, we are likely to see continued growth in hybrid mixes of structural material and solutions. New-build is a fraction of the building stock in use. There will be a rapidly increasing interest in refurbishment, dismantling and reuse of buildings. A material that, over the last 200 years, has shown itself to be supremely adaptable and consistent in its material properties once integrated into a building, steel is the material of choice for the discerning structural designer.

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18.11.2 Useful websites

For all steel components and whole building design information:
www.steelconstruction.org
www.steel-sci.com
www.tatasteelconstruction.com

doi: 10.1680/mosd.41448.0341

Chapter 19
Timber and wood-based products

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Eurocode 5 presents two very significant problems to timber designers in the UK. Firstly, it is a limit state code that will be superseding a permissible stress code of 60 years' standing, and secondly, there is a presumption that the designer will have reached an academic level equivalent to at least that of a first-year undergraduate, preferably in timber engineering. This chapter describes the basic properties of wood and wood-based products and their characteristics in different environmental conditions such as humidity and temperature, as well as describing the influence of duration of load. Design rules for ultimate and serviceability limit states are given. Fasteners such as nails, bolts and screws are described together with their design parameters. The design of the more common structural components, including vertical diaphragms, rules for designing plane frames and bracing, together with detailing requirements and certain controls applicable during construction are presented. This chapter covers the content of BS EN 1995-1-1 (*Design of Timber Structures: General – Common Rules and Rules for Buildings*), NA to BS EN1995-1-1 (*UK National Annex to Eurocode 5: Design of Timber Structures – Part 1-1: General – Common Rules and Rules for Buildings*), and PD 6693-1 (*Complementary Information to Eurocode 5: Design of Timber Structures Part 1 – General: Common Rules and Rules for Buildings*).

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19.1 Introduction

Methods of calculating the strength and stiffness of timber members have been in existence since the early 1800s, one method surprisingly using the plastic modulus for assessing the flexural strength of a rectangular member. The permissible stress code, CP112 *The Structural Use of Timber* was first published in 1952. The second revision in 1971 made provision for CP112-1 *Limit State Design, Materials and Workmanship* to be the forthcoming limit states design code. However, with the partial permanent and variable load factors of 1.4 and 1.6 respectively introduced in CP110 *Code of Practice for the Structural Use of Concrete* in 1972 and with the exclusion level used at that time for the statistical determination of timber strength properties (1 in 100), the unacceptable result was that the partial material factor, γM, had to be less than 1.0 to achieve parity with the member sizes given by the permissible stress code.

Further work on CP112-1 was deferred as in 1972 the Conseil International du Batiment Working Group 18 (CIB W18), an international timber research group, started drafting a timber limit states code and this was published in 1984. By this time CP112 had become BS 5268 *Structural Use of Timber* and the Eurocodes were being talked about and actively developed so the limit states part of the UK timber code never materialised. In 1988, the first draft of Eurocode 5: *Common Unified Rules for Timber Structures* was published.

This document was developed to give drafts for development of EC5 that were published in the UK in the 1990s:

DD ENV 1995-1-1:	*General – Common Rules and Rules for Buildings: 1994*
DD ENV 1995-1-2:	*General – Structural Fire Design: 1994*
DD ENV 1995-2:	*Bridges:1997*

In a design code encompassing the requirements of a large part of Europe, certain values used in the design procedures had to be delegated to national standards bodies for determination to allow for regional and traditional building variances. These were defined as 'boxed' values (later they became Nationally Determined Parameters, NDP). With the boxed values it was possible to carry out limit states design albeit using certain UK standards where necessary – loading, for example.

EC5 was developed further in the next 10 years, in particular attempting to conform to the harmonisation of the presentation of design procedures across all the material Eurocodes. BS EN1995 was published in 2004:

BS EN1995-1-1:	*General – Common Rules and Rules for Buildings (EC5-1-1)*
BS EN1995-1-2:	*General – Structural Fire Design (EC5-1-2)*
BS EN1995-2:	*Bridges (EC5-2)*

each part with its National Annex (NA) setting out the decisions made on the Nationally Determined Parameters. By virtue of the preliminary work done with the CIB W18 code, many of whose members were concerned with the drafting of EC5, there are relatively few NDPs in the three Parts of EC5 (EC5-1-1 has now 13 of which the UK NA departs only on 2 from the recommended decisions in the Eurocode). Corrigenda of EC5 have been and will be issued as necessary but major revisions are not scheduled to take place until at least 2015.

The various parts of BS 5268 included much practical, user-friendly advice and information on various aspects of timber construction, including the use of non-wood-based materials such as plasterboard for sheathing walls and ceilings, that will not be found in the wood-based EC5. There are also essential aspects surprisingly not in the Eurocode such as the design of glued joints. All this supplementary information, collectively known as Non-Contradictory Complementary Information (NCCI), is given in PD 6693-1 (*Complementary Information to Eurocode 5: Design of Timber Structures Part 1 – General: Common Rules and Rules for Buildings*).

The Eurocodes presume a minimum level of academic qualification so no mention will be found in EC5 of matters that would be presumed to have been taught in these studies. For example, the effective length of compression members in relation to the end conditions is not given in EC5 so this information is included in the NCCI.

At the same time as the Eurocode was being drafted, the necessary supporting standards were being prepared by the *Comité Européen de Normalisation* (CEN). CEN standards are written with each aspect of a particular material such as manufacture, grading, testing, marking, etc., being set out in a separate document whereas the corresponding British Standard would encompass all these facets in a single document. As a result there are numerically many more CEN standards referenced in EC5-1-1 than British Standards in BS 5268.

The EC5 design procedure starts with characteristic strength values with modification factors giving generally a lower design strength value whereas BS 5268 starts with the long term, lowest grade stress that may be factored upwards to achieve a higher permissible design stress. BS 5268 may therefore be considered ‘fail safe’ if modification factors are not used whereas EC5 is ‘fail unsafe’ if modification factors are not used.

A new code of practice – referring to new technology and with unfamiliar wording – is always a problem for the user. To compound this issue, EC5 will be an introduction to limit states design for many of its readers.

19.2 Timber and timber products

19.2.1 Introduction

Timber is available in many forms – softwood or hardwood either solid, laminated, veneered as in plywood and laminated veneered lumber (LVL) or reconstituted as in oriented strand board (OSB), particle board and fibreboard, usually in preferred sizes and formats as set out in European Standards or dictated by commercial production. The use of commonly available species and standard products provides an economic basis for design.

All these materials have defined mechanical properties for use in structural applications but the facility to ‘work’ a particular material can further influence choice. For example, certain hardwoods have excellent strength properties but the need to reset and resharpen tools more frequently can offset this attribute. Aesthetic factors such as surface texture and colour may also influence the choice of a particular material.

19.2.2 Solid timber

BS EN336 (*Structural Timber: Sizes, Permitted Deviations*) sets out dimensions and tolerances for sawn, regularised and ‘machined on four sides’ timber (**Table 19.1**). These sawn,

Size: mm	75	100	125	150	175	200	225	250	275	300
22		✔		✔	✔	✔	✔			
25	✔	✔	✔	✔	✔	✔	✔			
38	✔	✔	✔	✔	✔	✔	✔			
47	✔	✔	✔	✔	✔	✔	✔	✔		✔
63		✔	✔	✔	✔	✔	✔			
75		✔	✔	✔	✔	✔	✔	✔	✔	✔
100		✔		✔		✔	✔	✔		✔
150				✔		✔				✔
200						✔				
250								✔		
300										✔

Table 19.1 Preferred sizes for sawn timber (BSI, 2003). Permission to reproduce extracts from BS EN 336 is granted by BSI

regularised and machined on four sides dimensions are defined as ‘target sizes’ and any of the target sizes may be used in an EC5 calculation.

Surfaced or machined dimensions are achieved by subtracting 3 mm from sawn dimensions of 100 mm or less and 5 mm from the dimensions greater than 100 mm.

BS EN336 does not preclude the use of ‘non-standard’ dimensions for sawn, regularised or machined on four sides timber where such dimensions may be required for a particular purpose.

Tolerances are set out in BSEN 336 as follows:

> T1 is applied to sawn sizes and is –1 / +3 mm for dimensions 100 mm or less and –2 / +4 mm for dimensions greater than 100 mm.
>
> T2 is applied to surfaced or planed dimensions and is –1 / +1 mm and –1.5 / +1.5 mm for finished dimensions 100 mm or less and greater than 100 mm respectively.

So starting with a 100 × 47 sawn timber section the full dimensional specification would be 100(T1) × 47 (T1) for sawn timber, 97 (T2) × 47(T1) for timber regularised in the width and 97 (T2) × 44 (T2) for ‘machined on four sides’.

All these dimensional measurements are assumed to be at a moisture content of 20%. Adjustment for differences in dimension due to a moisture content different from 20% may be made.

In addition to the dimensions given above, there are sizes originating in North America – American Lumber Sizes (ALS) and Canadian Lumber Sizes (CLS) – that are surfaced on four sides with arris rounding (makes handling easier). The common sizes are 38 mm × 89 or 140 mm and many European mills also produce these sizes as they are widely used in house construction.

The length of a timber section is limited firstly by the tree size from which it is obtained and secondly by the presence of any unacceptable defects, as defined by the strength grading rules (see Section 19.4), that may have to be cut out. The lengths

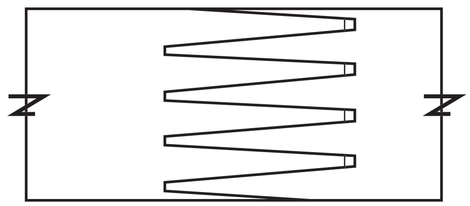


Figure 19.1 Finger joint (BSI, 2001). Permission to reproduce extracts from BS EN 385 is granted by BSI

then available can be joined together to form longer lengths by finger jointing (**Figure 19.1**) as described in BS EN385 (*Finger Jointed Structural Timber: Performance Requirements and Minimum Production Requirements*).

Finger jointed timber has been available in the UK since the 1960s with various finger joint profiles and adhesives giving strengths equivalent to the 'parent' wood.

19.2.3 Glued laminated timber and glued laminated solid timber

The manufacture and design of these products will be defined in a new BS EN14080 (*Timber Structures: Glued Laminated Timber and Glued Solid Timber – Requirements*) to be published in 2012.

This standard will describe *glued laminated timber* made from laminates of solid timber bonded under pressure using an appropriate adhesive (**Figure 19.2**). *Homogeneous glulam* is made from laminates of the same grade throughout whereas *combined glulam* is made with the outer laminates of a higher grade than the inner laminates. The proportion of inner laminates and their grading is variable and is not now confined to the old standard of 25% high grade in the outer zones and 50% lower grade in the centre.

The maximum thickness of laminate for straight sections is usually 45 mm when using softwoods; for above this thickness, cupping due to moisture effects (see Section 19.4) can prevent a full width adhesive bond. For curved sections the thickness of laminates may have to be reduced to allow for stresses arising within the laminate due to the bending of the timber to the required radius in addition to the stresses arising in the member as a whole.

For straight members the normal Scandinavian laminate sizes for manufacture are 45 mm thick with finished widths

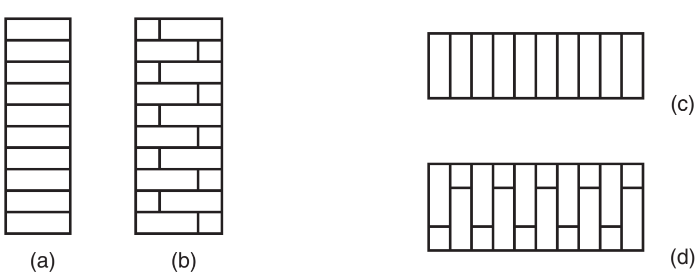


Figure 19.2 Glued laminated timber (BSI, in press). Permission to reproduce extracts from BS EN 14080 is granted by BSI

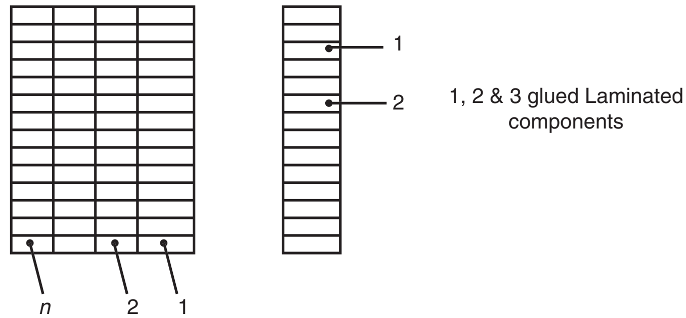


Figure 19.3 Glued members (BSI, in press). Permission to reproduce extracts from BS EN 14080 is granted by BSI

in 25 mm increments, for example, 90, 115, 140, etc. whereas the German, Austrian and other central European areas have a standard 40 mm thick laminate and finished widths in increments of 20 mm, for example, 100, 120, 140, etc. The maximum width of a single laminate is usually 250 mm so for wider beams either the layup shown in **Figure 19.2(b,d)** has to be used or glued members fabricated.

BS EN14080 also describes *glued members made from glued laminated members* – very large sections can pose problems in manufacture with regard to the time to spread adhesive on laminates in relation to the pot life of the adhesive or the member size may not 'fit' existing machinery. In this case, the final member may be built up from 'smaller' glued laminated members. This requires a gap filling adhesive to be used to bond the laminated components together and a means of holding the components together whilst the gap filling adhesive sets (screws, clamps, etc.).

BS EN14080 also describes *glued laminated solid timber* – essentially thicker laminates (up to 85 mm thick but not thinner than 45 mm) bonded in a manner similar to normal glulam but with restriction on the overall size of member to 280 × 240 mm. This is a specialist fabrication with limited use, usually for domestic constructions.

For all three forms of laminated construction the timber has to be kiln dried to a moisture content within 3% of the equilibrium moisture content that will be achieved in service which usually means drying to between 10% and 15% moisture content. The timber will then usually have to be finger jointed to achieve the required laminate length. The laminates then have to be surfaced to give flat, parallel bonding surfaces. There is usually a maximum time interval between this surfacing and gluing otherwise the cross-section may cup or the surfaces become contaminated thus reducing the bonding capacity. These surfaces are spread with an appropriate adhesive and the assembled laminated section then put under a pressure acting on the glue line of around 0.7 N/mm^2 (100 lbf/in^2) until the adhesive has set. The adhesives used in glulam manufacture require a minimum temperature of 10°C to set properly (a good reason not to contemplate site gluing!). This temperature is achieved by either ensuring the temperature of the wood is

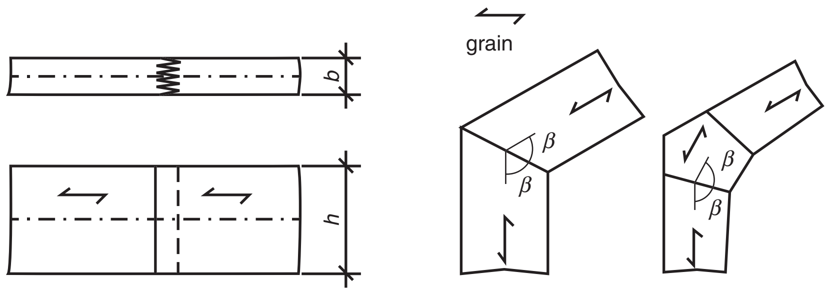


Figure 19.4 Large finger joints (BSI, 2001). Permission to reproduce extracts from BS EN 387 is granted by BSI

at least 10°C when gluing and the ambient temperature of the workplace is maintained at or above this level or the glueline may be set by radio frequency heating, in effect microwaving.

In the pressing process there will be some small misalignment of the laminates and squeeze out of adhesive that sets hard. The section then needs regularising to achieve the finished section. This regularising usually takes 10 mm off the initial breadth of the laminate. So a member with 10 laminates initially of size 100 × 45 mm at the time of pressing will produce a finished size of (100 – 10) × (10 × 45) or 90 × 450 mm say.

With the costs of drying the timber, specialised machines, a temperature controlled manufacturing space and adhesives, laminating is not a cheap process. The most basic straight rectangular laminated section costs in the order of three or four times the initial timber cost.

Large finger joints manufactured in accordance with BS EN 387 (*Glued Laminated Timber: Large Finger Joints – Performance Requirements and Minimum Production Requirements*) can be used to produce longer lengths of glulam and to change direction of members provided the angle β is not greater than 45º as shown in **Figure 19.4**.

19.2.4 Wood-based products

The best known wood-based product in structural applications is plywood. Today, it is losing prominence because of the shortage of logs for peeling the veneers to make the plywood. Most plywoods today are 'balanced' with an odd number of veneers so the two outer veneers have their face grain running in the same direction. Usually, the face grain runs parallel with the longest side, the exception being some Finnish plywoods. Plywoods are stronger in the face grain direction so the orientation of sheets for a particular design situation is often critical.

19.2.5 Engineered wood products

Laminated veneered lumber as described in BS EN 14279 (*Laminated Veneer Lumber (LVL) – Specification, Definitions, Classification and Requirements*) is in effect a uni-directional veneer plywood board. Some LVL layups have the occasional cross banding veneer at right angles to the body of the board that improves its dimensional stability. The mechanical properties are better than glulam. There are a number of sources of this material in the USA and Europe. To take due account of the size effect in the design of LVL in bending and tension to EC5, the manufacturer has to provide a size effect exponent, s, in the expression [3.3] of EC5-1-1 $(300/h)^s$ it typically can be taken as 0.12.

Although not featured in EC5, cross laminated timber (CLT or X-lam) is designed using EC5. Currently reliance has to be placed on manufacturer's details but draft standards for manufacture and design are in progress. CLT is similar to plywood in layup, with boards of thicknesses between 19 mm and 40 mm replacing the plywood veneers. The ends of boards in each layer are finger jointed with the longitudinal edge joints butted. The timber used is usually grade C24 so the section properties in each of the horizontal axes can be calculated using only the boards parallel to the axis under consideration, giving a two-way spanning component. The overall thicknesses, length and breadth are dependent on the manufacturer with smaller CLT members often factory bonded to give larger components. CLT is used for floor and roof panels, walls, columns and beams. The movement of CLT with moisture and temperature change is not too different from solid timber. Dowel type fasteners in CLT behave in a similar manner to solid timber.

Joists are an I-section with flanges of either solid timber or LVL and with an OSB/3 web. They are used mainly in roof and floor constructions but now wall thicknesses are increasing to accommodate thermal insulation they are finding growing application in walls. There is very little shrinkage in the depth of the joist after installation so it has particular application in multistorey platform frame constructions where moisture movement in the floor zones is critical. Care has to be taken to stabilise the narrow compression flange at all stages of construction.

19.3 The properties of timber and wood-based products

19.3.1 Introduction

An appreciation of the strength and deformation of timber components and buildings cannot be realised without a knowledge of the properties of the natural material, viz. strength grading of timber and wood-based materials, the orientation of actions in relation to the grain of the timber, the duration of actions and the effects environmental conditions have on timber and wood-based materials. The movement of timber and wood-based materials with time, varying moisture content and temperature change are also significant considerations in design. The following describes the influence of these factors together with the durability of timber and wood-based materials.

19.3.2 Grading of timber and wood-based products

All sawn timber used structurally should be traceable by the marking of each piece with an appropriate stamp describing the grade, the source of the material and how it was graded (visually or by machine with reference to an approved person or machine). If this is not possible, for example, for aesthetic reasons, then there should be documentation recording this information. The head standard for grading is BS EN 14081-1 (*Timber Structures: Strength Graded Structural Timber with Rectangular Cross Section – Part 1: General Requirements*) and for the visual grading of softwoods and hardwoods BS 4978 (*Visual Strength Grading of Softwood: Specification*) and BS 5756 (*Specification for Tropical Hardwoods for Structural Use*). Note that the traceability of the grading procedures also provides a step towards providing sustainable forestry and establishing the chain of certification.

Glued laminated timber is manufactured in accordance with BS EN14080 from laminates graded. Laminated veneer lumber (LVL) is manufactured in accordance with BS EN 14279.

Board materials such as particle board, oriented strand board (OSB), hardboard and plywood are manufactured to a particular standard for the material and grade all of which are identified in the standard for the strength values BS EN12369 (*Wood-Based Panels: Characteristic Values for Structural Design*).

19.3.3 The influence of grain direction on the properties of timber

Timber is an orthotropic material, i.e. it has different strength and stiffness values in each of the three principal axes – longitudinally parallel to the grain of the timber, perpendicular to the grain in the radial direction normal to the growth rings and perpendicular to the grain but tangential to the growth rings (**Figure 19.5**).

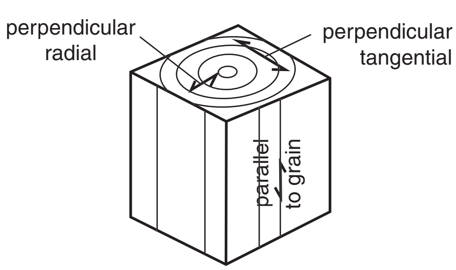


Figure 19.5 Grain directions in sawn timber

	Tension	Compression	Emean
Parallel to grain (0°)	14.0 N/mm²	21.0 N/mm²	11.0 kN/mm²
Perpendicular to grain (90°)	0.5 N/mm²	2.5 N/mm²	0.37 N/mm²

Table 19.2 Characteristic strengths of C24 timber at 0° and 90° to the grain (BSI, 2009). Permission to reproduce extracts from BS EN 338 is granted by BSI

In design there is no differentiation between the strengths radially and tangentially to the grain – a single value is given (**Table 19.2**).

These orientation differences are reflected also in board materials where the direction of the face grain veneers in plywood and the orientation of the flakes in oriented strand board influences strength and stiffness.

19.3.4 Moisture content and service classes

Timber and wood-based materials are moisture sensitive. Expressed very simply, solid timber is comprised of a series of interconnecting cells and when a tree is felled there is water in the cells and in the cell walls. The ratio of the weight of water in the timber to the weight of oven dried wood fibre (the moisture content usually expressed as a percentage) is in excess of 100% at this stage. When the timber is dried, the free water in the cells is lost first until at a particular moisture content (known as the *fibre saturation point*) any further drying takes moisture from the cell walls. Typically the fibre saturation point is around 25% and drying below this causes shrinkage The next moisture content threshold for solid timber is 20% – above this the timber is 'wet', liable to decay, difficult to glue and has lower tabulated strength and stiffness values.

Moisture classifications are defined in both BS 5268 and EC5 as Service Classes. Timber with a moisture content over 20% is in Service Class 3. Between 12% and 20% in Service Class 2 and 12% and below in Service Class 1. Examples of these Service Classes in a building are given in **Table 19.3** (taken from the UK National Annex Table NA.2).

There is no differentiation between SC1 and SC2 regarding strength properties but they do affect creep deflection.

Type of construction	Service class
Cold roofs	2
Warm roofs	1
Intermediate floors	1
Ground floors	2
Timber-frame walls – internal and party walls	1
– external walls	2
External uses where member is protected from direct wetting	2
External uses, fully exposed	3

Table 19.3 Service classes (BSI, 2004). Permission to reproduce extracts from UK NA BS EN 1995-1-1 is granted by BSI

Figure 19.6 Moisture effects on a section of timber

As solid timber changes its moisture content, the cross-section changes dimension. The movements are greater in the tangential direction than in the radial direction. This can result in 'cupping' of a timber section as shown in **Figure 19.6**. For practical purposes, the mean of the tangential and radial movements is used in assessing moisture movements in a building.

For the common softwoods (redwood, whitewood) the cross grain movement is often taken as 1% dimensional change for 4% change in moisture content so a 200 mm floor joist drying from 20% to 12% will likely shrink 2% of 200 = 4 mm. Note that the shrinkage of timber along its length or along the grain is so small that it is usually ignored (about 1/40th of the cross grain movement). For moisture movements of many hardwoods and softwoods see BRE (1982).

Shrinkage can be a major problem in multistorey platform frame constructions where at each floor the cross grain timber dimensions with solid timber floor joists could be 330 mm, i.e. a total of around 2000 mm of cross grain timber in a 7 storey building giving a potential shrinkage movement of 40 mm on drying from 20% to 12%. Add to this the elastic compression of the timber frame under gravitational loading of perhaps 5 mm and assuming brick cladding, a potential expansion of 18 mm (1 mm/m) gives a potential differential movement between masonry cladding and a timber frame window cill on the seventh floor of around 65 mm!

19.3.5 Temperature effects

Timber has low thermal conductivity (0.13 W/mK) and a low coefficient of linear thermal expansion (4×10^{-6} / ^{0}C). Expansion joints are not usually required in a timber building and these properties are also reasons why timber behaves well in the fire situation – disruptive expansion is minimised and the build up of temperature to the ignition point (about 300^{0}C) within a timber section is slow.

19.3.6 Duration of load

Timber is stronger the shorter the duration of an action as shown in **Figure 19.7** where the ratio of strength at time t seconds to that at permanent load duration expressed as a percentage = $(0.8192(\log_{10} t)^2 - 20.2390(\log_{10} t) + 225.3000)$.

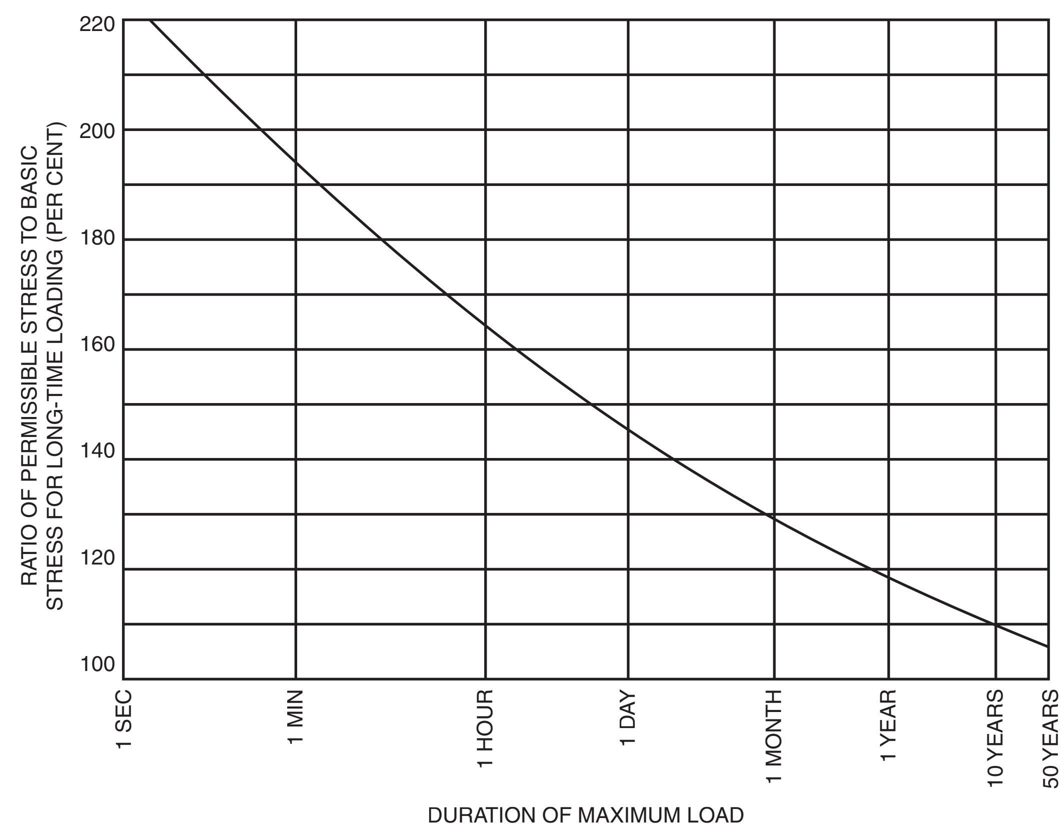


Figure 19.7 Relation of strength to duration of load (Booth and Reece, 1967). Reproduced courtesy of Spon

This relationship can be used to extrapolate beyond the limits of the curve, for example, very short durations such as blast loading and very long term loads where estimates of load effects can be made for durations of hundreds of years.

From the UK NA Table NA.1, the five durations of load for normal design conditions are:

permanent	say 50 years	self weight
long term	10 years	storage
medium term	6 months	floor imposed
short term	1 week	snow and residual structure after accidental loading
instantaneous		wind and accidental loading

19.3.7 Durability

Timber is susceptible to attack by fungi, insects, bacteria and marine borers. The greatest danger arises from water or moisture being in the wrong place within the structure leading to fungal decay. Attention has to be given at the design stage to ensure that surfaces shed water away from the face of the building, that condensation risks within the structure are eliminated and that ventilation either natural or forced removes unwanted moisture. In a properly designed building, preservative treatment is an insurance rather than protection against decay.

Use Classes 1, 2 and 3 of BS EN 335-1 (*Durability of Wood and Wood-Based Poducts – Definitions of Use Classes*) correspond with the three Service Classes with Use Class 4 relating to timber buried in the ground and use Class 5 to timber in sea water.

BS EN 350-2 (*Durability of Wood and Wood-Based Products: Natural Durability of Solid Wood*) has five durability classes – very durable, durable, moderately durable, slightly durable and not durable. The most commonly used softwoods fall in the slightly durable class. Certain hardwoods, such as greenheart, are very durable but they originate from the tropical rainforests and hence are costly and very difficult to 'work'. This standard also gives indications of how easily the various timbers can be treated – the common softwoods are classified as moderately easy to treat.

Table 19.4 of EC5 gives examples of corrosion protection to be provided for fasteners and metal plates used with timber structures. Care has to be taken with regard to the compatability of untreated timber, treated timber and the corrosion protection of fasteners and metal plates.

19.4 Mechanical properties of timber and wood-based materials

19.4.1 Characteristic strengths

The strength properties of materials are expressed as *characteristic values* at the 5th percentile exclusion level and therefore require modification to give *design values*. Stiffness parameters (E or G) are given as either mean values or 5th percentile values. The mean values are used for *estimating* deformations and the 5th percentile values are used essentially in stability calculations.

Solid timber properties are given in BS EN338 (*Structural Timber: Strength Classes*) for timber graded in accordance with BS EN14081. The grades such as C16 and C24 have been in BS 5268 for more than 15 years so timber graded and marked C24 can be used for both a BS 5268 design and an EC5 design, the only difference being the magnitudes of the tabulated strength properties (**Table 19.4**).

There is a significant difference between the BS 5268-2 and BS EN338 values for compression perpendicular to grain strengths. The characteristic value for C24 is 2.5 N/mm^2 which is only very slightly higher than the permissible long term stress of 2.4 N/mm^2 in BS 5268-2. This discrepancy is tempered in EC5-1-1 by the modifications described in clause 6.1.5.

19.4.2 Glued laminated timber

BS 5268-2 allows increasing strength values with increasing numbers of laminates but then effectively cancels this out with a decreasing depth factor. In EC5-1-1, there is no increase for numbers of laminates and only a modest increase for depths or widths less than 600 mm down to 230 mm. The layups (i.e. grade of laminate to give a glulam grade) and the glulam strength properties are currently given in BS EN1194 (to be replaced shortly by BS EN14080) (**Tables 19.5**, **19.6**, **19.7**).

Note that in middle Europe GL32h is usually made from C35 laminates. The the density values given for both grades of glulam are characteristic values appropriate for use in the design of connections. For the assessment of self weight the average density value should be used. As a rough guide the average density is 1.2 times the characteristic density.

In the UK, the highest common grade of softwood is TR26 (near enough C27) but BS EN1194 considers laminates as high as grade C40. The paradox is that a characteristic glulam bending strength of 32 N/mm^2 needs laminates with a characteristic bending strength of 35 or 40 N/mm^2 whereas there has been an understanding in the UK since the days of CP112-2 that glulam would give a higher strength than the strength of its component laminates.

The BS 5268 glulam strengths with laminate grading of C27 or higher originally gave unrealistically higher strength values than BS EN1194 but this was rectified in the 2007 revision of BS 5268-2.

19.4.3 Wood-based products

The characteristic strength for these boards are given in BS EN12369-1 & 2. An extract for OSB/3 is given in **Table 19.8**.

For other wood-based products reference has to be made to manufacturers data and European Technical Approvals.

	Poplar and softwood species												Hardwood species					
	C14	C16	C18	C20	C22	C24	C27	C30	C35	C40	C45	C50	D30	D35	D40	D50	D60	D70
Strength properties (N/mm²)																		
Bending	14	16	18	20	22	24	27	30	35	40	45	50	30	35	40	50	60	70
Tension parallel	8	10	11	12	13	14	16	18	21	24	27	30	18	21	24	30	36	42
Tension perpendicular	0.4	0.5	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Compression	16	17	18	19	20	21	22	23	25	26	27	29	23	25	26	29	32	34
Parallel compression	2.0	2.2	2.2	2.3	2.4	2.5	2.6	2.7	2.8	2.9	3.1	3.2	8.0	8.4	8.8	9.7	10.5	13.5
Perpendicular shear	1.7	1.8	2.0	2.2	2.4	2.5	2.8	3.0	3.4	3.8	3.8	3.8	3.0	3.4	3.8	3.8	3.8	3.8
Stiffness properties (kN/mm²)																		
Mean modulus of elasticity parallel	7	8	9	9.5	10	11	11.5	12	13	14	15	16	10	10	11	14	17	20
5% modulus of elasticity parallel	4.7	5.4	6.0	6.4	6.7	7.4	7.7	8.0	8.7	9.4	10.0	10.7	8.0	8.7	9.4	11.8	14.3	16.8
Mean modulus of elasticity perpendicular	0.23	0.27	0.30	0.32	0.33	0.37	0.38	0.40	0.43	0.47	0.50	0.53	0.64	0.69	0.75	0.93	1.13	1.33
Mean shear modulus	0.44	0.5	0.56	0.59	0.63	0.69	0.72	0.75	0.81	0.88	0.94	1.00	0.60	0.65	0.70	0.88	1.06	1.25
Density (in k/m³)																		
Density	290	310	320	330	340	350	370	380	400	420	440	460	530	560	590	650	700	900
Mean density	350	370	380	390	410	420	450	460	480	500	520	550	640	670	700	710	840	1080

NOTES:
1. Values given above for tension strength, compression strength, shear strength, 5% modulus of elasticity, mean modulus of elasticity perpendicular to grain and mean shear modulus, have been calculated using the equations given in clause Annex A.
2. The tabulated properties are compatible with timber at a moisture content consistent with a temperature of 20°C and a relative humidity of 65%.
3. Timber conforming to classes C45 and C50 may not be readily available.

Table 19.4 Strength classes – characteristic values (BSI, 2009). Permission to reproduce extracts from BS EN 338 is granted by BSI

Glulam strenth classes	GL 24	GL 28	GL 32
Homogeneous glulam	C24	C30	C40
Combined glulam: Outer/ inner laminations	C24/C18	C30/C24	C40/C30

Table 19.5 Typical glulam lay ups (BSI, 1999). Permission to reproduce extracts from BS EN 1194 is granted by BSI

19.5 Design rules for the ultimate limit states

19.5.1 Design strength value

The design strength of a particular property is given by

$$X_d = k_{mod} \frac{X_k}{\gamma_M} k_{various} \qquad [2.14]$$

This expression is applicable to material strengths, fastener loads, etc.

where X_d is the design value

k_{mod} is the factor for a particular type of material in a particular service class and for a particular duration of actions

X_k is the characteristic strength

γ_M is the partial material factor

$k_{various}$ modification factors for dimensions of members, factors for compression, shear, buckling, etc.

This expression is applicable to material strengths, fastener loads, etc.

Note that unlike BS 5268 where failure to use the modification factor for duration of load is 'fail safe', failure to use the appropriate EC5 modification factor k_{mod} will usually lead to a 'fail unsafe' situation as k_{mod} in many instances is less than 1.00.

The duration of load effect is combined with the service class in the factor k_{mod}.

Glulam strength class		GL 24h	GL 28h	GL 32h	GL 36h
Bending strength	$f_{m,g,k}$	24	28	32	36
Tension strength	$f_{t,0,g,k}$	16.5	19.5	22.5	26
	$f_{t,90,g,k}$	0.4	0.45	0.5	0.6
Compression strength	$f_{c,0,g,k}$	24	26.5	29	31
	$f_{c,90,g,k}$	2.7	3.0	3.3	3.6
Shear strength	$f_{v,g,k}$	2.7	3.2	3.8	4.8
Modulus of elasticity	$E_{0,g,mean}$	11 600	12 600	13 700	14 700
	$E_{0,g,05}$	9400	10 200	11 100	11 900
	$E_{90,g,mean}$	390	420	460	490
Shear modulus	$G_{g,mean}$	720	780	850	910
Density	$\rho_{g,k}$	380	410	430	450

Table 19.6 Characteristic values for homogeneous glulam (BSI, 1999). Permission to reproduce extracts from BS EN 1194 is granted by BSI

Glulam strength class		GL 24c	GL 28c	GL 32c	GL 36c
Bending strength	$f_{m,g,k}$	24	28	32	36
Tension strength	$f_{t,0,g,k}$	14	16.5	19.5	22.5
	$f_{t,90,g,k}$	0.35	0.4	0.45	0.5
Compression strength	$f_{c,0,g,k}$	21	24	26.5	29
	$f_{c,90,g,k}$	2.4	2.7	3.0	3.3
Shear strength	$f_{v,g,k}$	2.2	2.7	3.2	3.8
Modulus of elasticity	$E_{0,g,mean}$	11 600	12 600	13 700	14 700
	$E_{0,g,05}$	9400	10 200	11 100	11 900
	$E_{90,g,mean}$	320	390	420	460
Shear modulus	$G_{g,mean}$	660	720	780	850
Density	$\rho_{g,k}$	350	380	410	430

Table 19.7 Characteristic values for combined glulam (BSI, 1999). Permission to reproduce extracts from BS EN 1194 is granted by BSI

Note that many of the board materials do not have values for Service Class 3 indicating that they should not be used in this environment.

Where a single grade of material is used the k_{mod} value is given in **Table 19.9**. Where two different materials are used, for example, a nail through OSB/3 into C16 softwood then

$$k_{mod} = \sqrt{k_{mod1}k_{mod2}}$$

The possibility of unfavourable deviations of the material or product property from its characteristic value is covered by the partial material factor γ_M as given in **Table 19.10** taken from the UK NA Table NA.3.

Modification factors for the dimensions of members are:

k_h the depth factor in bending or width factor in tension members

$$=\left(\frac{150}{h}\right)^{0.2}$$ for rectangular solid timber in the range $1.0 \geq k_h \leq 1.3$ i.e. for depths 150 mm to 40 mm

$$=\left(\frac{600}{h}\right)^{0.1}$$ for rectangular glulam in the range $1.0 \geq k_h \leq 1.1$ i.e. for depths 600 mm to 235 mm

k_h for LVL in bending

$$=\left(\frac{300}{h}\right)^{s}$$ for rectangular LVL in the range $1.0 \geq k_h \leq 1.2$ i.e. for depths 300 mm to 65 mm

Note that the index s may be taken as 0.12 but should be checked against manufacturer's information.

k_l is the length factor for LVL in tension

$$=\left(\frac{3000}{l}\right)^{s/2}$$ for rectangular LVL in the range $1.0 \geq k_l \leq 1.1$, i.e. for lengths 3000 mm to 600 mm

Thickness, mm	Characteristic density (kg/m³) and strength (N/mm²) values: Density	Bending		Tension		Compression		Panel Shear	Planar Shear
k_{mod}	ρ	f_m		f_t		f_c		f_v	f_r
		0	90	0	90	0	90		
> 6 to 10	550	18.0	9.0	9.9	7.2	15.9	12.9	6.8	1.0
> 10 to 18	550	16.4	8.2	9.4	7.0	15.4	12.7	6.8	1.0
> 18 to 25	550	14.8	7.4	9.0	6.8	14.8	12.4	6.8	1.0

Thickness, mm	Mean stiffness values (N/mm²): Bending		` Tension		Compression		Panel Shear	Planar Shear
k_{mod}	E_m		E_t		E_c		G_v	G_r
	0	90	0	90	0	90		
> 6 to 10	4930	1980	3800	3000	3800	3000	1080	50
> 10 to 18	4930	1980	3800	3000	3800	3000	1080	50
> 18 to 25	4930	1980	3800	3000	3800	3000	1080	50

Table 19.8 Characteristic strength values for OSB/3 (BSI, 2001). Permission to reproduce extracts from BS EN 12369-1 is granted by BSI

Material	Standard	Service class	Load duration class				
			Permanent action	Long term action	Medium term action	Short term action	Instantaneous action
Solid timber	EN 14081-1	1	0.60	0.70	0.80	0.90	1.10
		2	0.60	0.70	0.80	0.90	1.10
		3	0.50	0.55	0.65	0.70	0.90
Glued laminated timber	EN 14080	1	0.60	0.70	0.80	0.90	1.10
		2	0.60	0.70	0.80	0.90	1.10
		3	0.50	0.55	0.65	0.70	0.90
LVL	EN 14374, EN 14279	1	0.60	0.70	0.80	0.90	1.10
		2	0.60	0.70	0.80	0.90	1.10
		3	0.50	0.55	0.65	0.70	0.90
Plywood	EN 636						
	Part 1, Part 2, Part 3	1	0.60	0.70	0.80	0.90	1.10
	Part 2, Part 3	2	0.60	0.70	0.80	0.90	1.10
	Part 3	3	0.50	0.55	0.65	0.70	0.90
OSB	EN 300						
	OSB/2	1	0.30	0.45	0.65	0.85	1.10
	OSB/3, OSB/4	1	0.40	0.50	0.70	0.90	1.10
	OSB/3, OSB/4	2	0.30	0.40	0.55	0.70	0.90
Particle-board	EN 312						
	Part 4, Part 5	1	0.30	0.45	0.65	0.85	1.10
	Part 5	2	0.20	0.30	0.45	0.60	0.80
	Part 6, Part 7	1	0.40	0.50	0.70	0.90	1.10
	Part 7	2	0.30	0.40	0.55	0.70	0.90
Fibreboard, hard	EN 622-2						
	HB.LA, H8.HLA 1 or 2	1	0.30	0.45	0.65	0.85	1.10
	HB.HLA 1 or 2	2	0.20	0.30	0.45	0.60	0.80
Fibreboard, medium	EN 622-3						
	MBH.LA 1 or 2	1	0.20	0.40	0.60	0.80	1.10
	MBH.HLS 1 or 2	1	0.20	0.40	0.60	0.80	1.10
	MBH.HLS 1 or 2	2	–	–	–	0.45	0.80
Fibreboard, MDF	EN 622-5						
	MDF.LA, MDF.HLS	1	0.20	0.40	0.60	0.80	1.10
	MDF.HLS	2	–	–	–	0.45	0.80

Table 19.9 Values of k_{mod} (BSI, 2004). Permission to reproduce extracts from BS EN 1995-1-1 is granted by BSI

Note that the index *s* may be taken as 0.12 but should be checked against manufacturer's information

k_{sys} is a load sharing factor normally 1.1 for spacing of members ≤ 610 mm centres where there is distribution of the applied loading by tiling battens, roof or floor decking across a number of members; a relaxation is given allowing spacings to be increased to 1200 mm dependent upon the stiffness of the load distributing elements.

Modification factors for structural effects such as instability are given in the following.

19.5.2 Member axes

The axes in all Eurocodes are expressed in the same manner as in a 3D computer analysis as shown in **Figure 19.8**, where the z axis is the minor axis.

19.5.3 Bending

The actual bending stress must be less than or equal to the design strength (**Figure 19.9**). Taking a simple beam bending about its major axis,

$$\text{bending stress } \sigma_{m,y,d} = \frac{M_{y,d}}{W_y} \leq \text{bending strength } f_{m,y,d}$$

$$= \frac{k_{mod}\ f_{m,k}\ k_{various}}{\gamma_M}$$

For biaxial bending, the bending stresses are $\sigma_{m,y,d} = \dfrac{M_y}{W_y}$ and $\sigma_{m,z,d} = \dfrac{M_z}{W_z}$. In a rectangular section allowance is made for local yield at P under the maximum stress accumulation by the factor k_m

with the combination of stresses limited by:

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \text{ or } k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0$$

[6.11] and [6.12]

Fundamental combinations:		
Solid timber	– untreated	1.3
	– Preservative treated	1.3
Glued laminated timber		1.25
LYL, plywood, OSB		1.2
Particle board		1.3
Fibreboard	– Hard	1.3
	– Medium	1.3
	– MDF	1.3
	– Soft	1.3
Connections (except for/punched metal plate fasteners)		1.3
Punched metal plate fasteners		
	– Anchorage strength	1.3
	– Plate (steel) strength	1.15
Accidental combinations		1.0

Table 19.10 Values of γ_M (BSI, 2004). Permission to reproduce extracts from UK NA BS EN 1995-1-1 is granted by BSI

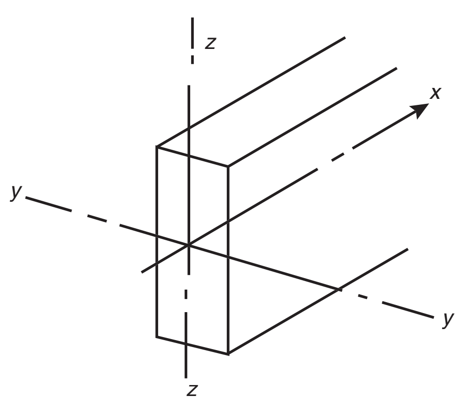


Figure 19.8 Member axes

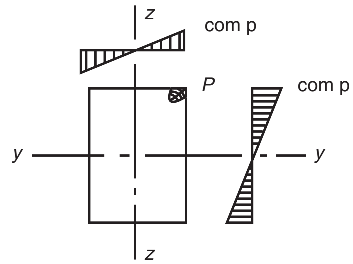


Figure 19.9 Accumulation of stress at a corner

where k_m = 0.7 for rectangular sections of solid timber, glulam and LVL

= 1.0 for all other shapes and materials. EC5 6.1.6(2).

The effect of end notches on bending close to the support of a beam is given in EC5 6.5.1.

19.5.4 Flexural members subject to bending or combined bending and compression (lateral instability)

The sequence of design steps for members subject only to bending may be simplified as follows:

(a) determine $\sigma_{m,\,crit}$ either from expression [6.31] or the simplification [6.32] (*note* the simplification relies on the ratio of E / G being 16 which is only applicable to softwoods but the approximation is close enough for glulam and hardwoods).

The simplification gives $\sigma_{m,\,crit} = \dfrac{0.78b^2}{h\ l_{ef}}$.

where b = breadth of the section
h = depth of the section
l_{ef} is the effective length derived from **Table 19.11**.

(b) from [6.30] $\lambda_{rel,m} = \sqrt{\dfrac{f_{m,k}}{\sigma_{m,crit}}}$

from [6.34] $k_{crit} = 1.00$ for $\lambda_{rel,m} \leq 0.75$
$= 1.56 - 0.75\ \lambda_{rel,m}$ for $0.75 < \lambda_{rel,m} \leq 1.4$
$= \dfrac{1}{\lambda^2_{rel,m}}$ for $1.4 < \lambda_{rel,m}$

Note that if the displacement of the compression edge is fully restrained then k_{crit} = 1.00.

Beam type	Loading type	l_{ef}/l[a]
Simply supported	Constant moment Uniformly distributed load Concentrated force at the middle of the span	1.0 0.9 0.8
Cantilever	Uniformly distributed load Concentrated force at the free end	0.5 0.8

[a] The ratio between the effective length l_{ef} and the span l is valid for a beam with torsionally restrained supports and loaded at the centre of gravity. If the load is applied at the compression edge of the beam, l_{ef} should be increased by 2h and may be decreased by 0.5h for a load at the tension edge of the beam.

Table 19.11 Effective length as a ratio of the span (BSI, 2004). Permission to reproduce extracts from BS EN 1995-1-1 is granted by BSI

(c) the actual stress must be less than the modified design strength [6.33]

$$\sigma_{m,y,d} \le k_{crit}\, f_{m,y,d}$$

Note that for members such as roof beams subject to gravitational actions with the top, compression edge laterally restrained by decking, purlins, etc. (k_{crit} = 1.0) wind suctions may cause reversal of stress and compression on an unrestrained bottom edge with k_{crit} becoming critical.

For combined bending and compression expression [6.35] must be met

$$\left[\frac{\sigma_{m,d}}{k_{crit} f_{m,d}}\right]^2 + \frac{\sigma_{c,0,d}}{k_{c,z}\, f_{c,0,d}} \le 1.0$$

where $k_{c,z}$ is given by [6.26].

19.5.5 Shear

Concentrated loads closer to the support than the effective depth of the member can be ignored for shear but not for bearing at the support. The breadth of a shear section has to modified by k_{cr} = 0.67 (UK NA Table NA.4) for solid timber and glulam otherwise k_{cr} = 1.0. In a full rectangular cross-section of solid timber or glulam the shear stress is given by $\dfrac{1.5V}{k_{cr}\ b\, h}$ [cl. 6.1.7].

EC5-1-1 cl. 6.5.2 gives the shear strength reduction for both over and under notched ends in solid timber, glulam and LVL (**Figure 19.10**).

For beams notched on the same side as the support (**Figure 19.9(a)**) (under notch)

i is the notch inclination; for a 90° notch i = 0

h is the beam depth

h_{ef} is the effective depth

x is the distance from the line of action of the support reaction to the corner of the notch

$$\alpha = \frac{h_{ef}}{h}$$

k_n is 4.5 for LVL, 5.0 for solid timber, 6.5 for glued laminated timber

$$k_v = \text{minimum of } 1 \text{ or} \left[\frac{k_n\left(1+\dfrac{1.1\,i^{1.5}}{\sqrt{h}}\right)}{\sqrt{h}\left(\sqrt{\alpha[1-\alpha]}+0.8\dfrac{X}{h}\sqrt{\dfrac{1}{\alpha}-\alpha^2}\right)}\right] \quad [6.62]$$

For a top notch (**Figure 19.10(b)**) k_v = 1.00

Taking a rectangular section under notched LVL beam with h_{ef} =100, h = 150, x = 50, l = 0 so α = 0.67 and k_n = 4.5 for LVL then

from [6.62] k_v = min. 1.0 or

$$\frac{4.5}{\sqrt{150}\left[\sqrt{0.67(1-0.67)}+0.8\times\dfrac{50}{150}\sqrt{\dfrac{1}{0.67}-0.67^2}\right]} = 0.49$$

Maximum shear strength is 0.49 $f_{v,d}$ with the shear stress = 1.5 V / (k_{cr} × b × h_{ef}).

19.5.6 Compression perpendicular to grain

To compensate for the low characteristic value for compression perpendicular to grain, the breadth of bearing is increased by 30 mm in a direction parallel to the grain where this dimension is available (see **Figure 19.11**) to give an effective bearing area A_{ef}.

In addition, the design strength $f_{c,90,d}$ can be increased by the factor $k_{c,90}$. EC5 is not clear on the values that may be ascribed to $k_{c,90}$. Together with the values given in PD 6693-1 $k_{c,90\ m}$ay be taken as:

(i) bearing of beams, joists and similar members

1.50 for softwood

1.75 for softwood glued laminated timber

(ii) studs bearing on a continuously supported plate

1.25 for softwood plate

1.50 for softwood glulam plate

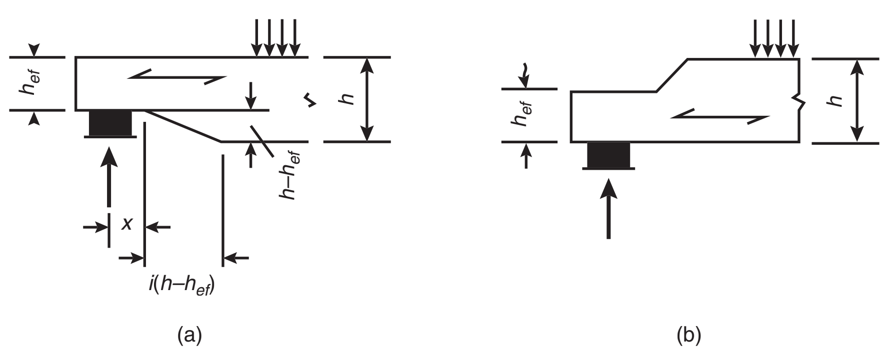


Figure 19.10 End notched beams (BSI, 2004). Permission to reproduce extracts from BS EN 1995-1-1 is granted by BSI

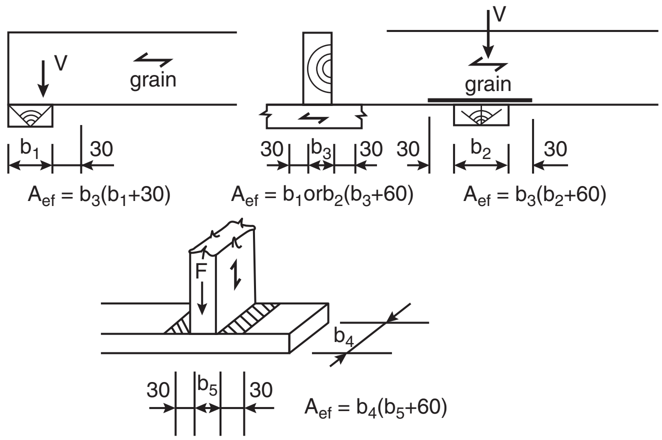


Figure 19.11 Effective bearing areas for compression perpendicular to grain

for the bottom chord of trussed rafters adjacent to the support where punched metal plates reinforce the bearing 1.50 for solid softwoods

There are no values quoted for hardwoods.

19.5.7 Compression members

The sequence of design steps for compression members given in EC5 may be simplified as follows:

(a) determine the effective lengths λ_y and λ_z using the factors in **Table 19.12** taken from PD 6693-1

$$\lambda_{rel.y} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}} \text{ and similarly } \lambda_{rel,z} \qquad [6.21]\ [6.22]$$

(b) if $\lambda_{rel,y}$ and $\lambda_{rel,z}$ are both < 0.3 the compression member is 'stocky' and the expressions below must be satisfied.

$$\left\{\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right\}^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m\frac{\sigma_{m,z,d}}{f_{m,z,d}} \qquad [6.19]$$

End condition	Ratio of effective length to actual length
Restrained at both ends in position and in direction	0.7
Restrained at both ends in position and at one end in direction	0.85
Restrained at both ends in position but not in direction	1.0
Restrained at one end in position and in direction and at the other end in direction but not in position	1.5
Restrained at one end in position and in direction and free at the other end	2.0

Table 19.12 Effective lengths of compression members (BSI, 2004). Permission to reproduce extracts from BS EN 1995-1-1 is granted by BSI

and

$$\left\{\frac{\sigma_{c,o,d}}{f_{c,o,d}}\right\}^2 + k_m\frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \qquad [6.20]$$

where k_m has the same values as for biaxial bending.

(c) otherwise buckling has to be considered.

$$k_y = 0.5(1 + \beta_c(\lambda_{rel,y} - 0.3) + \lambda^2_{rel,y}) \qquad [6.27]$$

$$k_z = 0.5(1 + \beta_c(\lambda_{rel,y} - 0.3) + \lambda^2_{rel,z}) \qquad [6.28]$$

where β_c is a 'straightness' factor (see EC5-1–1 cl. 10.2 (1)) [6.29]

= 0.2 for solid timber

= 0.1 for glulam and LVL

$$k_{c,y} = \frac{1}{k_{\gamma} + \sqrt{k_y^2 - \lambda^2_{rel,y}}} \qquad [6.25]$$

$$k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda^2_{rel,z}}} \qquad [6.26]$$

(d) so for combined axial and bending stresses

$$\frac{\sigma_{c,o,d}}{k_{c,y}f_{c,o,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m\frac{\sigma_{m,x,d}}{f_{m,x,d}} \le 1.0 \qquad [6.23]$$

and

$$\frac{\sigma_{c,o,d}}{k_{c,z}f_{c,o,d}} + k_m\frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,x,d}}{f_{m,x,d}} \le 1.0 \qquad [6.24]$$

19.5.8 Tension

Tension parallel to the grain is straightforward taking into account the size factor k_h for solid timber, glulam and LVL. In addition, LVL has a length based tension factor.

Tension perpendicular to the grain has to take into account the size of the member and the factor k_{vol}. The only example of this in EC5 is in the design of double tapered, circular and pitched cambered beams (see 6.10).

19.5.9 Torsion

$$\tau_{tor,d} \le k_{shape}f_{v,d} \qquad [6.14]$$

where k_{shape} = 1.2 for a circular cross-section

$$= \min 1.0 + 0.15\frac{h}{b} \qquad [6.15]$$

or 2.0 for a rectangular cross-section

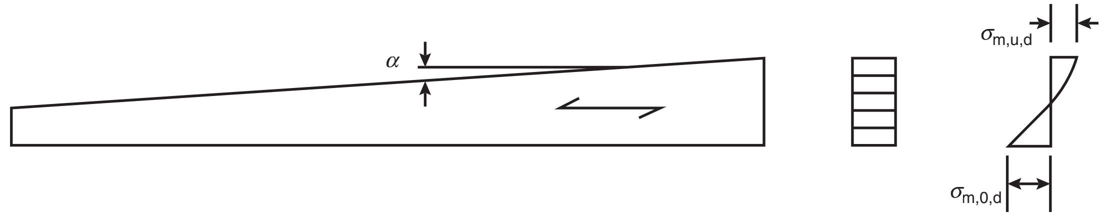


Figure 19.12 Tapered beam stresses (BSI, 2004). Permission to reproduce extracts from BS EN 1995-1-1 is granted by BSI

19.5.10 Members with varying cross section or curved shapes

The stress distributions in these beams are complex.

The simplest condition is the tapered beam (**Figure 19.12**) where the stresses parallel to the tapered edge are given in expressions [6.39] and [6.40].

The reduction factors for bending stress for various pitch angles are as shown in **Table 19.13**.

Note that the tension reduction is for the taper on the tension face and the compression reduction where the compression edge is tapered.

For double tapered beams, curved and pitched cambered beams the geometry may have to be determined by trial for conditions such as the radius may be critical (**Figure 19.13**).

The sequence of design steps given in EC5 may be simplified as follows.

(a) For curved and pitched cambered beams the strength of a section in the curved zone is limited to

$$k_r\, f_{m,d} \qquad [6.41]$$

where k_r = 1.0 if internal radius r_{in} / laminate thickness $t \geq 240$

or $0.76 + 0.001 r_{in} / t$ [6.49]

for double tapered beams $k_r = 1.0$

(b) At the apex the bending stress

$$\sigma_{m,d} = k_1 \frac{6M_{apex}}{bh_{ap}^2} \qquad [6.42]$$

Taper angle	Tension	Compression
2.5°	0.88	0.97
5.0°	0.66	0.89
7.5°	0.47	0.78
10.0°	0.34	0.68

Table 19.13 Reduction factors for bending stress at the tapered face (BSI, 2012). Permission to reproduce extracts from PD 6693-1 is granted by BSI

where

$$k_l = k_1 + k_2\left(\frac{h_{ap}}{r}\right) + k_3\left(\frac{h_{ap}}{r}\right)^2 + k_4\left(\frac{h_{ap}}{r}\right)^3 \qquad [6.43]$$

and $k_1 = 1 + 1.4 \tan\alpha_{ap} + 5.4 \tan^2\alpha_{ap}$ [6.44]

$k_2 = 0.35 - 8 \tan\alpha_{ap}$ [6.45]

$k_3 = 0.6 + 8.3 \tan\alpha_{ap} - 7.8 \tan^2\alpha_{ap}$ [6.46]

$k_4 = 6 \tan^2\alpha_{ap}$ [6.47]

$r = r_{in} + 0.5\, h_{ap}$ [6.48]

(c) In the beams shown in **Figure 19.12** tensile stresses perpendicular to the grain will be created in the apex zone under gravitational loading; this tensile stress is limited to

$$\sigma_{t,90,d} \leq k_{dis}\, k_{vol}\, f_{t,90,d} \qquad [6.50]$$

where k_{vol} = 1.0 for solid timber

$= \left(\frac{V_0}{V}\right)^{0.2}$ for glued laminated timber and LVL [6.51]

with V_0 = 0.01 m³

V = the volume of the apex zone (shown hatched in **Figure 19.12**) but not greater than 2/3 of the total volume of the beam

k_{dis} = 1.4 for double tapered and curved beams [6.52]

= 1.7 for pitched cambered beams

and

$$\sigma_{t,90,d} = k_p \frac{6M_{ap,d}}{b\, h_{ap}^2} \qquad [6.54]$$

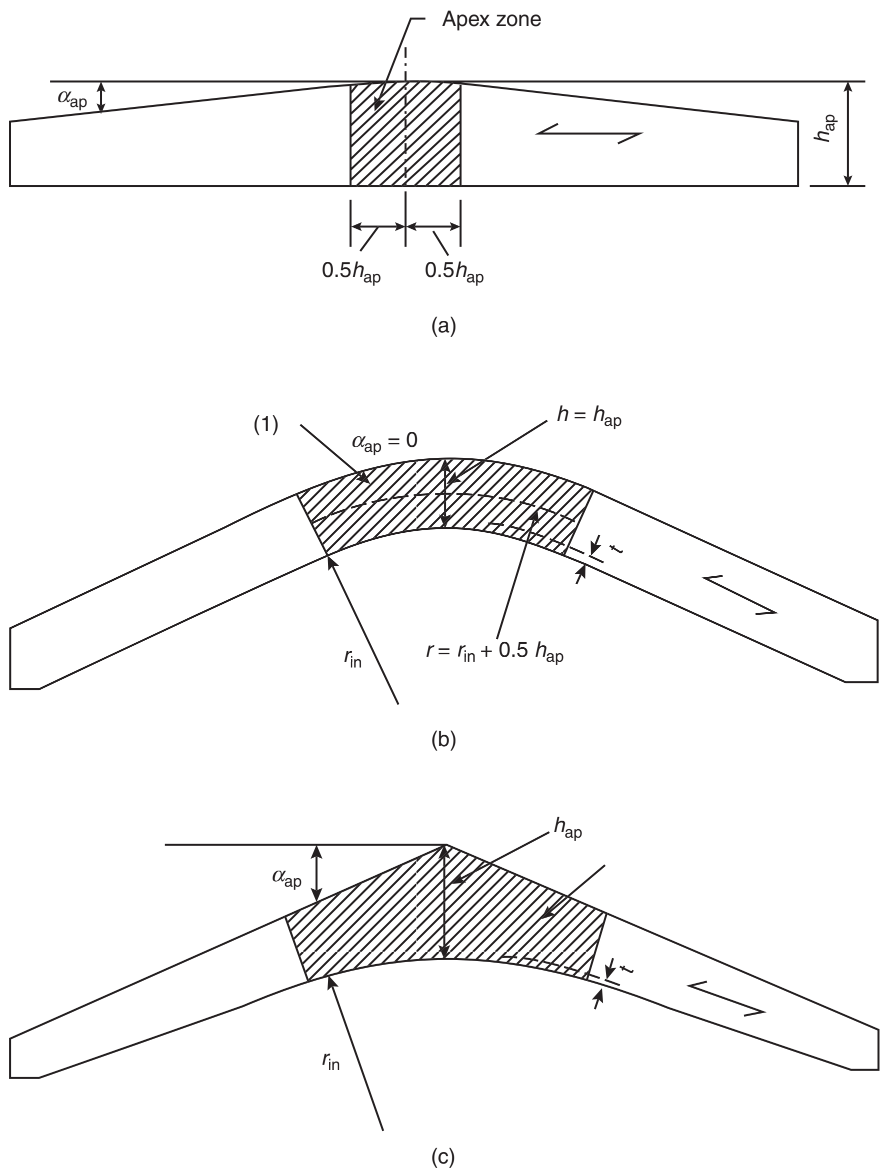
Note that this is a UK NDP decision.

$$k_p = k_5 + k_6\left(\frac{h_{ap}}{r}\right) + k_7\left(\frac{h_{ap}}{r}\right)^2 \qquad [6.56]$$

and $k_5 = 0.2 \tan\alpha_{ap}$ [6.57]

$k_6 = 0.25 - 1.5 \tan\alpha_{ap} + 2.6 \tan^2\alpha_{ap}$ [6.58]

$k_7 = 2.1 \tan\alpha_{ap} - 4.0 \tan^2\alpha_{ap}$ [6.59]



NOTE: In curved and pitched cambered beams the apex zone extends over the curved parts of the beam.

Figure 19.13 (a) Double tapered beam; (b) curved beam; (c) pitched cambered beam (BSI, in press). Permission to reproduce extracts from BS EN 14080 is granted by BSI

Note that the tensile stress perpendicular to the grain may require radial reinforcement usually in the form of bolts, glued in rods or screws.

(d) For combined tension perpendicular to the grain and shear in the apex zone

$$\frac{\tau_d}{f_{v,d}} + \frac{\sigma_{t,90,d}}{k_{dis}\ k_{vol}\ f_{t,90,d}} \leq 1.0 \qquad [6.53]$$

Glulam, with a profile similar to the pitched cambered beam, is often used for three pinned arch construction. Under gravitational loading the stress perpendicular to the grain in the 'apex' zone is then compression which may reverse to tension under wind loads. The principal practical problem is transporting the half span member from the factory to the site. Note that the decision to use [6.54] is UK NA 2.4.

19.6 Design rules serviceability

19.6.1 Serviceability limit states (SLS)

There are basically three serviceability states that have to be met.

(a) Limiting excessive deformation that could affect appearance, comfort of the user and function of the structure that could cause damage to finishes and non-structural members.

(b) Limiting damage that could affect the functioning, durability and appearance of the structure.

(c) Limiting vibration that could cause discomfort to the user and restrict the use of the structure.

19.6.2 Deformations

EC5 limits the deformations described in 7.1.1a) and b) by setting three action conditions as described in BS EN1990 6.5.3:

(a) characteristic combination for situations such as cracking that will not reverse on unloading, standing water that could lead to premature failure, etc.

$$\sum_{j\geq 1} G_{k,j} + Q_{k,1} + \sum_{i>1} \psi_{0,i}Q_{k,i}$$

(b) frequent combination for reversible situations such as deflection of lintels, deformations of beams, etc.

$$\sum_{j\geq 1} G_{k,j} + \psi_{1,1}Q_{k,1} + \sum_{i>1} \psi_{2,i}Q_{k,i}$$

(c) quasi-permanent combination used for creep deformations

$$\sum_{j\geq 1} G_{k,j} + \sum_{i\geq 1} \psi_{2,i}Q_{k,i}$$

Whilst Chapter 7 of EC5 is headed 'Serviceability limit states' the basic requirements are set out earlier in section 2.2.3. The instantaneous deformation is calculated using the actions given in 7.1.2 a) and the creep deformation is calculated using the actions given in 7.1.2 c) multiplied by the creep factor k_{def}. This gives the values in section 2.2.3 of EC5-1-1.

The final deformation, $u_{fin,G}$, i.e. including creep and joint slip, under permanent actions is given as

$$u_{fin,G} = u_{inst,G}\,(1 + k_{def}) \qquad [2.3]$$

where $u_{inst,G}$ is the instantaneous deformation and k_{def} is the long term creep factor for timber and wood-based materials in different Service Classes

similarly $u_{fin,Q1} = u_{inst,Q1}\,(1 + \psi_{2,Q1}k_{def})$ for leading variable action Q_1 [2.4]

and $u_{fin,Qi} = u_{inst,Qi}\,(\psi_{0,i} + \psi_{2,Q1}k_{def})$ for variable actions Q_i $(i > 1)$ [2.5]

hence $u_{fin} = u_{fin,G} + u_{fin,Q1} + \sum_{i\geq 1} u_{fin,Qi}$ for total deformation [2.2]

EC5 uses the mean values of E and G (EC5-1-1 2.2.3(2)) for deformation calculations of all members with no differentiation put on principal members and E_{min} and G_{min} as in BS 5268. Values of k_{def} are given in **Table 19.14**.

The total deflection including creep may be compensated for by an inbuilt camber. The intention of a camber is to cancel out the deflection due to permanent action and creep ideally leaving a small residual camber (see **Figure 19.14**).

Cambers are usually only achievable with glulam. It is common practice to provide a camber of twice the instantaneous deflection due to permanent actions.

The limiting deflections are given in the UK NA as set out in **Table 19.15**.

Material	Standard	Service class 1	Service class 2	Service class 3
Solid timber	EN 14084-1	0.60	0.80	2.00
Glued laminated timber	EN 14080	0.60	0.80	2.00
LVL	EN 14374, EN 14279	0.60	0.80	2.00
Plywood	EN 636			
	Part 1	0.80	–	–
	Part 2	0.80	1.00	–
	Part 3	0.80	1.00	2.50
OSB	EN 300			
	OSB/2	2.25	–	–
	OSB/3, OSB/4	1.50	2.25	–
Particle board	EN 312			
	Part 4	2.25	–	–
	Part 5	2.25	3.00	–
	Part 6	1.50	–	–
	Part 7	1.50	2.25	–
Fibreboard, hard	EN 622-2			
	HB.LA	2.25	–	–
	HB.HLA1 HB.HLA2	2.25	3.00	–
Fibreboard, medium	EN 622-3			
	MBH.LA1, MBH.LA2	3.00	–	–
	MBH.HLS1, MBH.HLS2	3.00	4.00	–
Fibreboard, MDF	EN 622-5			
	MDF.LA	2.25	–	
	MDF HLS	24.25	3.00	–

Table 19.14 Values of k_{def} (BSI, 2004). Permission to reproduce extracts from BS EN 1995-1-1 is granted by BSI

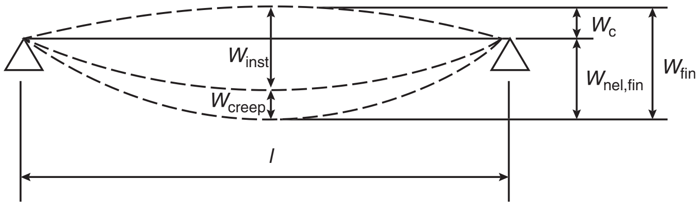


Figure 19.14 Elements of deflection (BSI, 2004). Permission to reproduce extracts from BS EN 1995-1-1 is granted by BSI

19.6.3 Vibration of residential floors

EC5-1-1 describes domestic floors that are essentially two-way spanning (joists and counter battens). It was found impossible to adapt the theory set out in EC5-1-1 to UK floors which are essentially one-way spanning.

In the UK NA 2.7, the concentrated load procedure of EC5-1-1 is developed for one-way spanning domestic floors comprised of solid timber joists, I-joists and metal web joists. Extensive testing of floors with all three types of joist has led to expression NA.1 of the National Annex. A 1.0 kN concentrated load applied to a joist in a floor construction will be distributed between the loaded joist and the adjacent joists depending on the stiffnesses of the transverse elements such as floor decking, ceiling and in the case of metal web joists 'strong backs' or transverse members within the depth of the joist. It has been known that with a solid timber joist floor only about 50% of the concentrated action is acts on the loaded joist. With the procedure described in the National Annex this proportion can be as low as 30%.

Table 19.16 gives the limits set out in the National Annex for the deflection limit *a* and the impulse velocity constant *b* given in [7.3] and [7.4].

The fundamental frequency, f_1, of the floor should not be less than 8 Hz as given by expression [7.5] of EC5.

$$f_1 = \frac{\pi}{2l^2}\sqrt{\frac{(EI_l)}{m}} \geq 8.0\,\text{Hz}$$

Type of member	Limiting value for net final deflections of individual beams, $\omega_{net,fin}$	
	A member of span, *l* between two supports	A member with a cantilever, *l*
Roof or floor members with a plastered or plasterboard ceiling	*l*/250	*l*/125
Roof or floor members without a plastered or plasterboard ceiling	*l*/150	*l*/75

NOTE When calculating $\omega_{net,\,fin,}$ ω_{fin} should be calculated as u_{fin} in accordance with BS EN 1995-1-1:2004 + A1:2008, 2.2.3(5).

Table 19.15 Limiting values for deflection (BSI, 2004). Permission to reproduce extracts from UK NA BS EN 1995-1-1 is granted by BSI

Parameter	Limit	
a, deflection of floor under a 1 kN point load	1.8 mm	for *l* ≤ 4000 mm
	16 500/$l^{1.1}$ mm	for *l* > 4000 mm
	where *l* = joist span in mm	
b, constant for the control of unit impulse velocity response	for *a* ≤ 1 mm for *a* > 1 mm	*b* = 180–60α *b* = 160–40α

NOTE The formulae for b correspond to BS EN1995-1-1:2004 + A1:2008, Figure 7.2. With a value of 0.02 for the modal damping ratio, ζ, the unit impulse velocity response will not normally govern the size of floor joists in residential timber floors.

Table 19.16 Limits for *a* and *b* (BSI, 2004). Permission to reproduce extracts from UK NA BS EN 1995-1-1 is granted by BSI

where m is the mass of the floor in kg/m² ignoring partitions and variable actions

l is the floor span in m

EI_l is the plate bending stiffness in the joist direction in Nm²/m which can be derived including the effects of decking, ceiling and where relevant strong backs.

The footnote to **Table 19.16** implies that unit impulse velocity will not be critical if the value of *b* is used together with a modal damping ratio $\xi = 0.02$. Advantage can be taken of the stiffening effects of floor decking, ceiling and where relevant strong backs in [7.7] to determine n_{40} in expression [7.6].

19.7 Metal fasteners and glued joints

19.7.1 Design criteria

The arrangement and sizes of fasteners in a connection including the fastener spacings edge and end distances give the required strength and stiffness. Due account has to be taken of the number of fasteners in a connection and the possibility that the resultant strength could be lower than the sum of individual strengths.

In multiple shear plane connections, the minimum resistance of each shear plane should be calculated on the assumption that the shear plane is a component of a three member connection. The modes of failure across the connection as a whole should be compatible; this precludes failure modes where failure is in bearing.

The strength capacity of a connection has to be reduced where it is subject to alternating long and medium term member forces. The strength of the connection should be either the tensile force + 50% of the compression force or vice versa.

The possibility of splitting caused by tension perpendicular to the grain shall be considered. In **Figure 19.15**, the maximum of $F_{v,Ed,1}$ and $F_{v,Ed,2}$ should be less than the design splitting capacity $F_{90,Rd}$. For softwoods

$$F_{90,Rk} = 1.4bw\sqrt{\frac{h_e}{\left(1-\frac{h_e}{h}\right)}} \qquad [8.4]$$

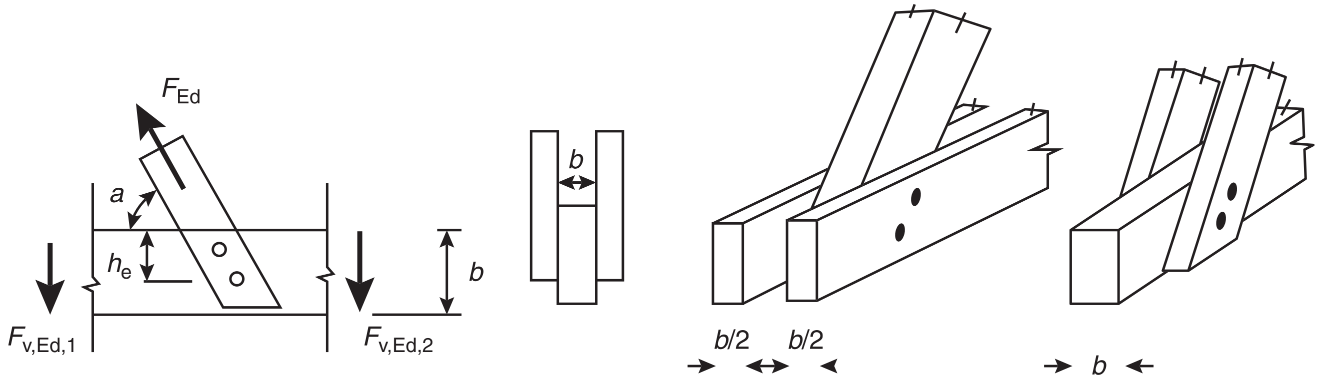


Figure 19.15 Inclined forces acting on a connection (BSI, 2004). Permission to reproduce extracts from BS EN 1995-1-1 is granted by BSI

where $w = \max \left(\frac{w_{pl}}{100}\right)^{0.35}$ or 1.00 for punched metal plate fasteners

or 1.00 for all other fasteners [8.5]

w_{pl} is the width of the punched metal plate fastener parallel to the grain in mm,

and

$$F_{90,Rd} \frac{k_{mod}F_{90,k}}{\gamma_M}$$

19.7.2 Dowel type fasteners

The basic theory of dowel type fasteners was developed by K. W. Johansen and taken further at Brighton University. The bearing and yield patterns for two and three member timber-to-timber joints and panel to timber joints are shown in **Figure 19.16** and these are related to the relevant expressions in **Figure 19.17**.

Johansen's formulae assume a 'frictionless' dowel that can slip in the fastener hole under the forces applied to the joint. Nails (particularly deformed nails such as annular ring shank), screws and bolts with the prescribed washers are not 'frictionless' and give an enhancement to the Johansen effect. This enhancement was proved for bolts at Nottingham University when the Johansen component alone was found some 30% lower than the equivalent BS 5268 values!

Note that there is a limit to this enhancement for each type of fastener, for example, staples 0%, plain round wire nails 15%, other nails 50%, screws 100%, bolts 25% and dowels 0% of Johansen.

Similar failure modes and formulae are given for combinations of steel plates and timber (**Figure 19.17**). Steel plates are classified as 'thin' if the plate thickness is less than 0.5 fastener diameter and 'thick' if more than one fastener diameter. For plates between 'thin' and 'thick', interpolation of the formulae is required.

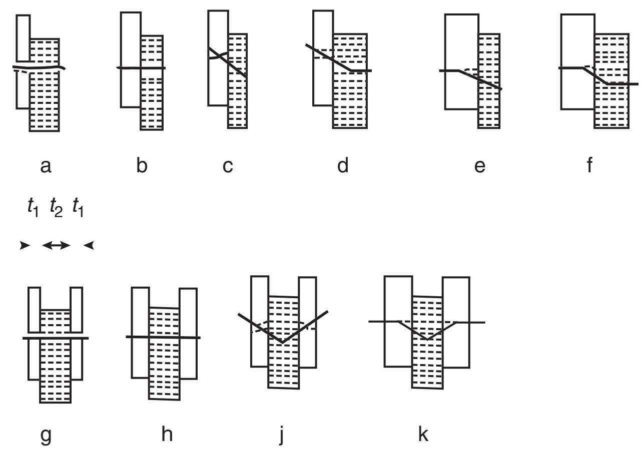


Figure 19.16 Failure modes in 2 and 3 member joints made with steel plates (BSI, 2004). Permission to reproduce extracts from BS EN 1995-1-1 is granted by BSI

For a thin steel plate in single shear:

$$F_{y,Rk} = \min \begin{cases} 0.4 f_{h,k} t_1 d & \text{(a)} \\ 1{,}15\sqrt{2M_{y,Rk} f_{h,k} d} + \dfrac{F_{ax,Rk}}{4} & \text{(b)} \end{cases}$$

For a thick steel plate in single shear:

$$F_{y,Rk} = \min \begin{cases} f_{h,k} t_1 d & \text{(c)} \\ f_{h,k} t_1 d \left[\sqrt{2 + \dfrac{4M_{y,Rk}}{f_{h,k} d t_1^2}} - 1\right] + \dfrac{F_{ax,Rk}}{4} & \text{(d)} \\ 2{,}3\sqrt{M_{y,Rk} f_{h,k} d} + \dfrac{F_{ax,Rk}}{4} & \text{(e)} \end{cases}$$

For a steel plate of any thickness as the central member of a double shear connection:

$$
F_{\mathrm{r,Rk}} = \min \begin{cases}
f_{h_2 1,k} t_2 d & \text{(a)} \\
f_{h,2,k} t_2 d & \text{(b)} \\
\dfrac{f_{\mathrm{h,l,k}} t_1 d}{1+\beta}\left[\sqrt{\beta + 2\beta^2\left[1+\dfrac{t_2}{t_1}+\left(\dfrac{t_2}{t_1}\right)^2\right]+\beta^3\left(\dfrac{t_2}{t_1}\right)^2} - \beta\left(1+\dfrac{t_2}{t_1}\right)\right] + \dfrac{F_{\mathrm{ax,Rk}}}{4} & \text{(c)} \\
1.05\dfrac{f_{\mathrm{h,l,k}} t_1 d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\dfrac{4\beta(2+\beta)M_{\mathrm{y,Rk}}}{f_{\mathrm{h,l,k}}\, d\, t_1^2}} - \beta\right] + \dfrac{F_{\mathrm{ax,Rk}}}{4} & \text{(d)} \\
1.05\dfrac{f_{\mathrm{h,l,k}} t_2 d}{1+2\beta}\left[\sqrt{2\beta^2(1+\beta)+\dfrac{4\beta(1+2\beta)M_{\mathrm{y,Rk}}}{f_{\mathrm{h,l,k}}\, d\, t_2^2}} - \beta\right] + \dfrac{F_{\mathrm{ax,Rk}}}{4} & \text{(e)} \\
1.15\sqrt{\dfrac{2\beta}{1+\beta}}\sqrt{2M_{\mathrm{y,Rk}} f_{\mathrm{h,l,k}} d} + \dfrac{F_{\mathrm{ax,Rk}}}{4} & \text{(f)}
\end{cases}
$$

$$
F_{\mathrm{r,Rk}} = \min \begin{cases}
f_{\mathrm{h,1,k}} t_1 d & \text{(g)} \\
0.5 f_{\mathrm{h,2,k}} t_2 d & \text{(h)} \\
1.05\dfrac{f_{\mathrm{h,l,k}} t_1 d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\dfrac{4\beta(2+\beta)M_{\mathrm{y,Rk}}}{f_{\mathrm{h,l,k}}\, d\, t_1^2}} - \beta\right] + \dfrac{F_{\mathrm{ax,Rk}}}{4} & \text{(j)} \\
1.15\sqrt{\dfrac{2\beta}{1+\beta}}\sqrt{2M_{\mathrm{y,Rk}} f_{\mathrm{h,l,k}} d} + \dfrac{F_{\mathrm{ax,Rk}}}{4} & \text{(k)}
\end{cases}
\qquad \text{with} \qquad \beta = \frac{f_{\mathrm{h,2,k}}}{f_{\mathrm{h,1,k}}}
$$

where

$F_{V,Rk}$ is the characteristic load carrying capacity per shear plane per fastener.
t_i is the timber thickness or penetration depth
$f_{h,i,k}$ is the characteristic embedment strength in timber i for the particular type of fastener
d is the fastener diameter
$M_{y,R,k}$ is the characteristic fastener yield moment
$F_{ax,Rk}$ is the characteristic axial withdrawal capacity of the fastener.

The first expression is the Johansen effect and the second, $F_{ax,Rk}/4$ is the 'rope effect'.

Figure 19.17 Characteristic shear values for 2 and 3 member and wood-based material joints (BSI, 2004). Permission to reproduce extracts from BS EN 1995-1-1 is granted by BSI

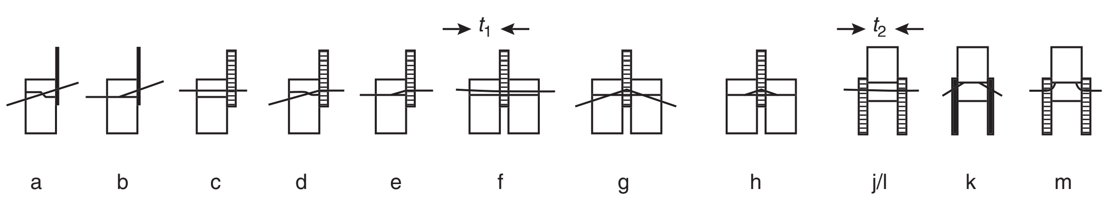


Figure 19.18 Characteristic shear values for steel plates in single and double shear (BSI, 2008). Permission to reproduce extracts from BS EN 14592 is granted by BSI

$$
F_{\mathrm{y,Rk}} = \min \begin{cases}
f_{\mathrm{h,l,k}} t_1 d & \text{(f)} \\
f_{\mathrm{h,l,k}}\left[\sqrt{2+\dfrac{4M_{\mathrm{y,Rk}}}{f_{\mathrm{h,l,k}} d t_1^2}} - 1\right] + \dfrac{F_{\mathrm{ax,Rk}}}{4} & \text{(g)} \\
2.3\sqrt{M_{\mathrm{y,Rk}} f_{\mathrm{h,l,k}} d} + \dfrac{F_{\mathrm{ax,Rk}}}{4} & \text{(h)}
\end{cases}
$$

For thick steel plates as the outer members of a double shear connection:

$$
F_{\mathrm{y,Rk}} = \min \begin{cases}
0.5 f_{\mathrm{h,l,k}} t_1 d & \text{(j)} \\
1.15\sqrt{2M_{\mathrm{y,Rk}} f_{\mathrm{h,2,k}} d} + \dfrac{F_{\mathrm{ax,Rk}}}{4} & \text{(k)}
\end{cases}
$$

For thick steel plates as the outer members of a double shear connection:

$$F_{\mathrm{y,Rk}} = \min \begin{cases} 0.5 f_{\mathrm{h,2,k}} t_2 d & \text{(l)} \\ 2.3\sqrt{M_{\mathrm{y,Rk}} f_{\mathrm{h,2,k}}}\, d + \dfrac{F_{\mathrm{ax,Rk}}}{4} & \text{(m)} \end{cases}$$

where

$F_{y,Rk}$ is the characteristic load carrying capacity per shear plane per fastener

$f_{h,i,k}$ is the characteristic embedment strength in the timber member

t_1 is the smaller of the thicknesses of the timber side member or the penetration depth

t_2 is the thickness of the timber middle member

d is the fastener diameter

$M_{y,R,k}$ is the characteristic fastener yield moment

$F_{ax,Rk}$ is the characteristic axial withdrawal capacity of the fastener

19.7.3 Shear strength values for dowel type fasteners

The procedure for arriving at the characteristic strength is similar for all dowel fasteners:

(a) Determine the characteristic yield moment for the fastener $M_{y,Rk}$ with the tensile strengths given in EC5-1-1 and also in the following for the particular type of fastener under consideration.

(b) Determine the embedment strength $f_{h,k}$ for the materials used – this is equivalent to the bearing strength of the fastener and can be determined either from the expressions given in EC5-1-1 or by test in accordance with BS EN383 (*Timber Structures: Test Methods – Determination of Embedment Strength and Foundation Values for Dowel Type Fasteners*).

From (a) and (b) the Johanssen expressions can be evaluated.

(c) Determine the characteristic withdrawal strength of the fastener which is the lower value of the withdrawal strength of the shank and the pull through value of the head or in the case of bolts the bearing strength of the washer.

From (c) the value of the 'rope effect' is obtained.

(d) Hence the characteristic shear strength of the fastener is given by the summation of the Johanssen value and the 'rope effect'.

Where two different wood-based materials are joined $\dfrac{k_{mod} = \sqrt{k_{mod1}\, k_{mod2}}}{\gamma_M}$ and $\gamma_M = 1.3$ in the derivation of the design strength from $\dfrac{k_{mod} F_{v,Rk}}{\gamma_M}$

More often than not sizes of members at a connection are dictated by jointing considerations.

19.7.4 Conditions for the use of dowel fasteners

19.7.4.1 Nails

- The equivalent diameter of square and grooved nails should be taken as the side dimension. (EC5-1-1 8.3.1.1(3))
- For nails 8 mm diameter or less the nail embedment values [8.15, 8.16, 8.20, 8.21, 8.22] should be used.
- Where the diameter is greater than 8 mm, the nail should be designed as a bolt. Note that it is recommended that the spacings, etc. for nails are used for every diameter of nail as the bolt spacings given in EC5-1-1 **Table 19.4** presume a 1 mm clearance hole not present when driving a nail even if pre-bored; this clearance reduces the effect of shrinkage and any tendency for the timber to split, hence the smaller spacings for bolts than for nails, for example, 4d edge distance for bolts compared with 7d for nails.
- A minimum of 2 nails should be used in a laterally loaded connection.
- Where the spacing of nails in a line parallel to the grain is less than 14d the cumulative capacity of a row of nails is less than the sum of each individual nail (EC5-1-1 8.3.1.1(8)) but it is acceptable to draw a line and drive nails alternately either side of the line without losing any cumulative capacity (EC5-1-1 Figure 8.6)!
- The minimum pointside penetration of a nail into timber should be at least 8d for smooth nails and 6d for other nails; the head diameter of the nail should be ≥ 2d.
- Timber should normally be pre-drilled
  where d > 8 mm or
  $\rho_k > 500$ kg/m^3 or
  t < max of 7d or (13d – 30) $\rho_k / 400$
- The spacings and edge distances for nails are given in **Table 19.2** of EC5-1-1.
- For panel products/timber joints the edge distances cannot be modified from **Table 19.2** but the spacings can be factored by 0.85.
- The edge distances for steel plate/timber joints cannot be modified from **Table 19.2** but the spacings can be factored by 0.70.
- Except in secondary structures smooth nails should not be used in shear in end grain. Other nails may be used provided the pointside penetration is at least 10d, at least 3 nails form a joint, the joint is in Service Class 1 or 2 and the shear strength is taken as 1/3 that of a smooth nail of corresponding diameter.
- Smooth nails should not be used to resist permanent or long term axial loading (medium, short and instantaneous axial loadings OK); only the threaded part of threaded nails should transmit axial force but EC5 then only gives design information relevant only to smooth nails, for example, the full calculated value for the axial load on a smooth nail applies where the pointside penetration is ≥ 12d with a linear reduction in strength down to zero at the minimum pointside penetration of 8d.
- EC5 gives advice on slant (toe) nailing and in particular the loaded edge distance (EC5-1-1 **Figure 8.8**).

The strength of a nail connection loaded in shear and axially is given in [8.27] and [8.28].

19.7.4.2 Staples

Staples are not widely used in the UK. Staples are effectively two separate nails with the following specific requirements:

- The equivalent diameter of rectangular section staples is $\sqrt{b \times h}$ where b and h are the staple dimensions.
- The width of the staple crown should be a minimum 6d and the pointside penetration 14d.
- There should be at least 2 staples in a connection.
- A staple may be calculated as 2 nails provided the angle between the line of the staple crown and the grain is more than 30° otherwise the shear capacity has to be down rated to 70% of the full capacity.
- The characteristic yield moment of each leg is given by $240d^{2.6}$ provided the wire has a minimum tensile strength of 800 N/mm^2.
- There is a reduction in strength for a series of staples in a line.
- Staple spacings differ from nails (EC5-1-1 **Table 19.3**).

19.7.4.3 Bolts

- The characteristic tensile strength for bolts is 400 N/mm^2 for grade 4.8 bolts and 800 N/mm^2 for grade 8.8.
- Bolt spacings and edge distances are given in EC5-1-1 **Table 19.4**.
- There is a reduction for the number of bolts lying in line parallel to the grain, for example, the capacity of 8 bolts in line at the standard spacing of 4d with the load parallel to grain is reduced to 4.8 bolts.
- To achieve the design strength, nuts and bolt heads must have the correct washers (EC5-1-1 cl. 10.4.3). The washer diameter should be 3 × bolt diameter and washer thickness 0.3d.
- For bolts the characteristic axial strength value $F_{ax,Rk}$ is net washer area × 3 × characteristic compression perpendicular to grain strength of the material under the washer.
- For external steel plates, an equivalent diameter of bearing is given by the smaller of 12 × plate thickness or 4 × fastener diameter.

19.7.4.4 Dowels

Dowels are designed in a similar manner to bolts but with lower spacings and edge distances as given in EC5-1-1 **Table 19.5**.

19.7.4.5 Screws

Section 8.7.1 of EC5-1-1 is not easy to follow. The following sets out the intentions of 8.7.1 in a more comprehensive manner.

(a) Screws should conform with the requirements of BS EN14592 (*Timber Structures: Fasteners – Requirements*).

(b) Screws may be formed by either

  (i) the threaded part being turned down from the original rod diameter such that the thread diameter, d, is equal to the smooth shank diameter (as given in BS 1210 *Specification for Wood Screws*) or

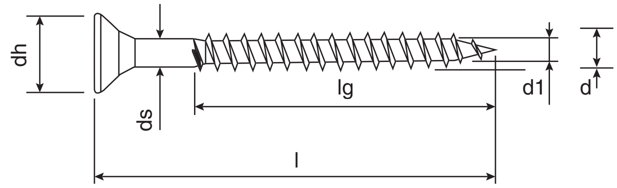


Figure 19.19 Geometry of a raised thread screw

  (ii) the threaded part being produced by rolling or forging such that the thread diameter, d, is greater than the smooth shank diameter (these screws are often referred to as 'raised thread screws' (**Figure 19.19**).

(c) The length of thread created may be either the full length of the screw or part length (raised thread screws with d greater than 6 mm are usually part threaded).

(d) The effective diameter d_{ef} of a screw should be taken as either

  (i) the smooth shank diameter (ds in **Figure 19.19**) where the smooth shank extends a distance of $4d_{ef}$ either side of the shear plane under consideration or

  (ii) 1.1 × root thread diameter where the condition described in (i) (above) does not exist.

  Note that for the screws defined in (ii) (above) it may be convenient to take d_{ef} as the smaller of the smooth shank diameter and 1.1 × root thread diameter (d1 in **Figure 19.19**) so the location of the shear plane relative to the shank and the threaded part is immaterial.

(e) The lateral load carrying capacity of screws should be calculated in accordance with Section 8.2 of BS EN1995-1-1 with the effective diameter d_{ef} being used instead of d in the expressions in 8.2.

(f) For screws with an effective diameter, d_{ef}, of 6 mm or less the lateral load carrying capacity should be calculated in accordance with the rules of 8.3.1 of BS EN1995-1-1 for nails otherwise the lateral load carrying capacity should be in accordance with the rules for bolts given in 8.5.1.

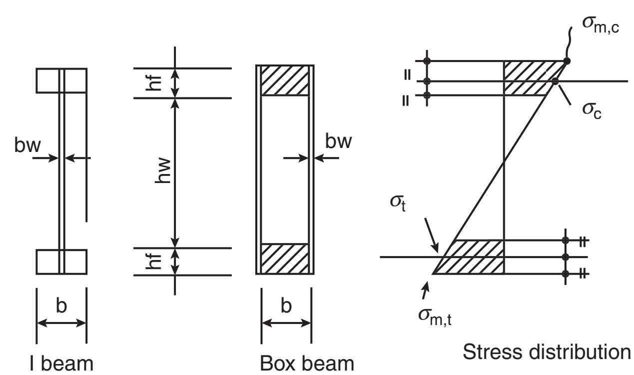


Figure 19.20 Thin webbed beams

(g) The embedment strength, $f_{h,k}$, and yield moment, $M_{y,k}$, should be calculated using d_{ef} *note* there could be an advantage in using the accredited values for yield moments given in a manufacturer's European Technical Approval, for the tensile strength is likely to be closer to 750 N/mm^2 than the 600 N/mm^2 for nails.

(h) The spacings and edge distances for all diameters of screws should be based on BS EN1995-1-1 **Table 19.2** using the thread diameter d for the same reasons set out for nails in 8.4.1.

For screws in withdrawal the strength of the thread may be readily calculated but EC5-1-1 refers to manufacturer's data for head pull through and the tensile strength of the shank. European Technical Approvals are now available from a number of manufacturers with this information which is very important as head pull through is usually the critical parameter in the characteristic withdrawal strength of a screw.

19.7.4.6 Punched metal plate fasteners (PMPF) and nail plates

PMPF rely on manufacturer's test data for assessment of characteristic design values. As a result it is difficult to run design checks as one might do for other fastener designs.

The strength of nailed steel plates of various forms and shapes, for example, tie straps, joist hangers, etc. can be calculated using the rules given in EC5-1-1 8.2 with due allowance for the close spacing of fasteners.

19.7.4.7 Connectors

A very large number of connectors are described in BS EN912 (*Timber Fasteners: Specification for Connectors for Timber*). Connectors familiar to the UK timber industry are

type A	split rings
type B	shear plates
type C	tooth plates *note* the types commonly used in the UK are C6, C7*, C8 and C9*. (* square sided)

EC 5-1-1 provides calculation procedures for connectors and does not rely on accumulated test data as does BS 5268-2.

The mode of failure of types A and B is by either shearing out the block of timber in front of the connector or if there is adequate end distance by embedment (compression) failure of the wood; the embedment failure may be either crushing or splitting of the timber. Hence the timber dimensions, the diameter of the connector, the spacing, end distances and the density of the timber are critical to the shear strength of the connector.

Tooth plate connectors, type C, fail primarily in embedment (compression) of both the teeth and the connector bolt. With small end distances, splitting and shearing out of a block of timber can occur. End distance is therefore critical with this type of connector. It is usual to use a high tensile bolt at least for the embedment process and then possibly replacing with a mild steel bolt. The strength of the fastener is the connector strength plus the bolt strength.

19.7.5 Joint slip

Slip is the local elastic deformation of a connection. This deformation is additional to the elongation or shortening of the members in a framework and any 'lack of fit' at the connection, for example, the 1 mm clearance usually provided when drilling holes for bolts. Slip and lack of fit can significantly increase the deformation of a pin jointed structure.

EC5-1-1 7.1 provides methods of estimating K_{ser} in N/mm for various types of fastener so given the axial force in a member the local deformation at the connection can be estimate. An iterative procedure is required to accommodate the effects of the varying of member forces and with slip.

Ideally the computer software used for frame analysis should be able to provide spring stiffness at the ends of members. Where the program does not have this facility, then short fictitious members have to be created at the ends of members having an appropriate E × Area to give the required elongation or shortening corresponding to the slip value.

19.7.6 Glued joints

Surprisingly EC5-1-1 does not mention glued joints. This omission is corrected in PD 6693-1. The range of adhesives now available is much greater than tabulated in BS 5268-2: for example, moisture cured polyurethanes, cross-linked polyvynyl acetate, melamine urea formaldehyde – used in laminating both as a conventional adhesive as a gap filling adhesive. Note that for normal gluing the glueline thickness should not be greater than 1 mm but there are situations where a modified 'gap filling' adhesive is required with a glueline thickness up to 3 mm.

Environmental conditions are as critical as with glued laminated timber with the temperature of the joint at fabrication and during setting and curing being critical, together with the moisture content of the bonding surfaces.

Shear strength of materials at the bonded surfaces are:

(a) timber to timber – the characteristic rolling shear strength of softwood at an angle α to the grain is
$f_{v,}\ \alpha_{,k} = f_{v,k}\ (1–0.67 \sin \alpha)$ where $f_{v,d}$ is the design shear strength parallel to the grain

Note that timber and wood-based products have fibres parallel to the grain and in very simple terms when a shear force is applied at an angle to the grain there is a tendency for the fibres to 'roll' over the fibres below hence the term 'rolling shear'.

(b) timber to plywood – the direction of grain alternates as the veneer orientation changes by 90° through the plywood thickness so rolling shear may be critical at the interface of timber and plywood or at the first glueline into the plywood depending on the layup of the plywood; the characteristic rolling shear strength is taken as the planar shear strength of the material c0 timber to OSB/3 – the rolling shear stress is calculated at the timber/panel interface with the characteristic rolling shear strength being the planar shear strength given in BS EN12369-1.

'Glued in rods' were originally included in EC5-2 and were then intended to be in EC5-1-1, with the unfortunate result that they do not now appear in either document. They are frequently used with the design based on the common sense application of the rules for movements of timber, stresses parallel and perpendicular to the grain and the use of adhesives.

19.8 Components, plane frames, bracing, detailing and control

19.8.1 Introduction

Solid timber, glulam, LVL and certain panel products can be used in mechanically connected or glued assemblies to create structural members either as 'one off' or batch production runs. In both cases, attention must be paid to economy of production by using where possible standard sizes of commonly available materials, straightforward assembly details with consideration given to handling, storage and installation of the member.

19.8.2 Thin web beams

Thin web beams today are epitomised by the mass-produced I joists used for roofs, floors and walls. They are constructed with either solid timber or LVL as the flanges and usually OSB/3 as the web although examples of plywood webs may be found in older constructions.

Section properties can be calculated by making allowance for the different stiffness values of the component materials.

The EC5-1-1 calculation procedure for flexural strength requires two checks – the first the bending stress at the outer fibres ($\sigma_{m,t}$ or $\sigma_{m,c}$) related to the bending strength of the flange material and the second axial tension / compression stress (σ_t or σ_c) at the mid depth of the flange related to the tension / compression strength of the flange. The latter condition will control in most circumstances – see **Figure 19.20**.

Checks should be made for:

(a) buckling of the compression flange particularly I beams during the construction stage.

(b) the web(s) for axial, shear and bending stresses as well as web buckling if $h_w > 70b_w$.
Note that for in plane bending of the web(s) the strength should be taken as the tension and compression strengths.

(c) the interface of the web and flange should be checked for rolling shear where the rolling shear stress is given by

$$= \frac{VA_f y_f}{nh_f I_{yy}}$$

where V = shear force
A_f = flange area
y_f = distance of centroid of flange from neutral axis
n = number of shear planes between flange and web
I_{yy} = second moment of area of section about y–y axis

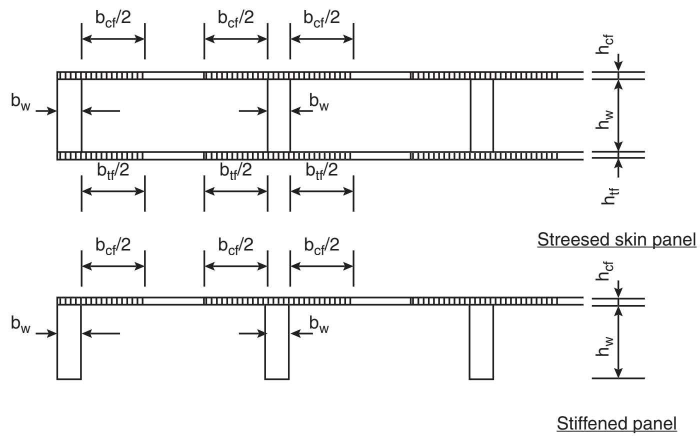


Figure 19.21 Stressed skin and stiffened panels

19.8.3 Thin flange beams

Thin flange beams with tension and compression flanges are known as 'stressed skin panels' and 'stiffened panels' with only the compression flange (see **Figure 19.21**).

Note that stressed skin panels act acoustically as a drum and unless appropriate measures are taken they are not recommended for floor constructions.

There is little practical experience of the use of materials other than plywood for the flanges.

The effective widths of the compression and tension flanges, b_{cf} and b_{tf}, assumed in the design of the I- and C-sections are set by consideration of shear lag and plate buckling.

Checks should be made for:

(a) the bending strengths of the flanges should be the tension and compression strengths of the material;

(b) the rolling shear stress between the flanges and webs;

(c) blockings between web members to provide torsional stability and a surface on which to join flange sheets.

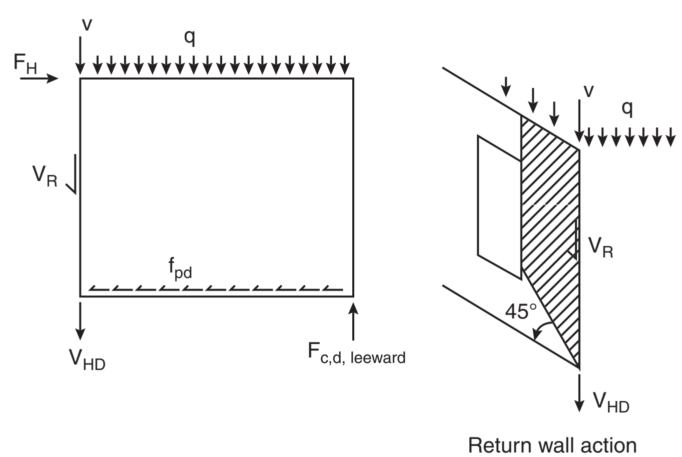


Figure 19.22 Wall type A

19.8.4 Mechanically jointed components

Attention must be paid to the influence of joint slip in any member that relies upon mechanically fasteners for its structural integrity.

Where the shear force varies along the length of a member, as in a beam, resulting in maximum and minimum spacing dimensions of fasteners, s_{max} and s_{min}, then provided $4s_{min} \geq s_{max}$, a uniform effective spacing of $s_{ef} = 0.75s_{min} + 0.25s_{max}$ may be used.

Note that the spacing of fasteners may be determined by the requirements of fixing one material to another e.g. the need to prevent a board material distorting.

Annexes B and C of BS EN1995-1-1 give methods of calculating mechanically jointed beams and columns.

19.8.5 Multi plane trusses

Bolted and connectored trusses have compression, tension and web members lying in different planes. The important consideration is to have a 'balanced' construction with an odd number of planes otherwise the truss will bow out plane due to eccentricity. For example, in the simplest triangular truss the rafters could lie in the outer planes (nos 1 and 3) and the tie in the centre (plane 2). This concept can be extended to 3, 5, 7 or even 9 planes.

BS EN1995-1-1 9.2.1 gives guidance on assessing effective lengths of members and the distribution of moments in, for example, continuous rafters. The principles of frame analysis for these components are given in BS EN1995-1-1 5.4.2.

19.8.6 Single plane trusses

Better known as trussed rafters, a simplified analytical procedure is given in BS EN1995-1-1 5.4.3 and design considerations in 9.2.2.

19.8.7 Diaphragms

It is difficult to produce rigid moment resistant joints in a timber frame so most structures rely on some form of horizontal bracing usually in the form of horizontal diaphragms to transmit wind forces and the like to vertical diaphragms and thence to the foundations. Horizontal diaphragms occur in many buildings either as roof or floor decking, roof sarking or ceilings and vertical diaphragms are created by wall sheathings such as OSB/3 and plasterboard.

Horizontal diaphragms can be created by careful arrangement of floor and roof decking sheets together with calculation and specification of the connection of the decking to the supporting members. Where reliance is placed solely on the decking to form the diaphragm the ratio of length to breadth of the diaphragm should not exceed 2.

Diaphragms with a length to breadth ratio of up to 6 can be designed with perimeter timber members creating the flanges of a horizontal web beam as described in BS EN1995-1-1 9.2.3.2(2).

The shear forces in the width of the diaphragm can be assumed to be uniformly distributed across the width. Where the sheets forming the diaphragm are laid in a block bonded pattern the spacing of fasteners in the width direction can be increased by a factor of 1.5 up to a maximum of 150 mm.

19.8.7.1 Vertical diaphragms – general

The first drafts of BS EN1995-1-1 contained what is now described as Method A for the design of vertical diaphragm panels to resist horizontal forces in the plane of the wall. This relied on the design strength of the fasteners connecting the wall sheathings to the bottom rail of the timber frame wall to take the horizontal force and required holding down connections at the ends of walls or where discontinuities occurred within the length of the wall due to door or window openings.

Despite being allowed on a statistical basis to increase the design strength of fasteners connecting the sheathing to the timber frame by a factor of 1.2, it was found that the racking shear strength with Method A was lower than that achieved using BS 5268-6.1 or 6.2 when account was taken of the differences between ultimate limit states design and permissible stress design. The holding down requirements of Method A are not normally required in a BS 5268 design.

The UK therefore submitted an alternative design procedure which is included in BS EN1995-1-1 as Method B. This is effectively the BS 5268 method converted to limit states.

Method B has problems with regard to acceptability in that the BS 5268 design method is based on the analysis of many hundreds of racking tests on various configurations of timber frame walls. The results are therefore not verifiable by calculation alone. Nevertheless it was included in BS EN 1995-1-1:2004.

A compromise was sought (named Method C!) and it was soon recognised that the omission in Method A of the contribution of the fasteners between the wall studs and the bottom rail of the timber frame panel was the 10% to 15% difference in racking strength between Methods A and B. A calculation procedure that allowed the elimination of the holding down requirements of Method A was also developed.

The design procedure for vertical diaphragms given in PD 6693-1 is Method C. It is relevant to storey height wall panels and where necessary utilises a uniformly distributed vertical connection force at the bottom edge of the diaphragm to eliminate requirement for specific holding down connections, provided that the fasteners in the residual length of the bottom edge of the diaphragm can resist the horizontal shear force.

A racking wall is built up from one or more wall diaphragms. The wall may be broken down into separate diaphragms by discontinuities created, for example, by doors and similar openings. Fully framed window openings may occur within a diaphragm albeit with a reduction in racking strength. Smaller openings with a dimension not more than 300 mm may occur in a diaphragm without reduction in strength.

Each diaphragm may be built up from one or more separate panels provided the vertical joint between panels is structurally connected. It is also good practice to tie the heads of panels together with a horizontal wall plate or to use the floor or roof construction bearing on the wall to provide this tie.

Each panel is built up from a timber frame of studs and horizontal rails lined on one or both faces with an acceptable structural material such as OSB/3 or plasterboard. It is allowable to have two sheathing layers on one face of the timber frame but then no allowance can be made for any sheathing on the other face. The sheathing may be applied as vertical or horizontal sheets fastened to the frame with a specified fastener configuration around the perimeter of a sheet, with usually the same fastener at twice the spacing for fixings to the frame within the perimeter of the sheet. There are restrictions on the minimum size of sheet that may be used as a racking sheathing.

The horizontal shear applied to the diaphragm is transmitted by the sheathings and their fastening to the bottom horizontal member of the timber frame. A horizontal shear connection has then to be made to the supporting structure by either mechanical fasteners or by friction between the timber and the structure it bears on or a combination thereof. PD 6693-1 gives a value of 0.4 for the coefficient of sliding friction between timber and wood-based products and other materials such as concrete, masonry, steel, etc. If reliance is placed solely on friction to take the horizontal shear then mechanical fixings should be provide to locate the diaphragm and also to resist forces normal the plane of the diaphragm.

Any vertical holding down forces have to be connected to the structure below whether a timber construction or the foundation and this substrate has to be capable of sustaining the uplift force.

19.8.7.2 Design of vertical diaphragms

The accumulated applied horizontal shear forces, F_H, acting on a diaphragm will create a destabilising moment acting about the leeward bottom corner of the diaphragm, countered by the permanent actions and any specific holding down, V_{HD}, provided acting about this point. **Figure 19.22** shows the action on a diaphragm that is held down only at the windward corner. The action V_R can come from the return wall and any forces acting on this wall. Any actions on the return wall within a 45° line drawn from the base of the wall can be considered contributing to V_R. For stability, due account has to be taken of whether actions are favourable or unfavourable and the appropriate partial load factor used.

The development of **Figure 19.22** is shown in **Figure 19.23** where the fasteners between the sheathings and the bottom rail are assumed resist uplift as well as horizontal shear. It may be necessary to supplement this holding down with the tie down force, V_{HD}, at the windward corner in order to have sufficient shear capacity in the remaining fasteners to resist the horizontal shear. The strength of the fastenings in uplift may be a proportion of the shear strength of the bottom rail fasteners, i.e. $f_{w,d} = uf_{p,d}$ particularly where the vertical connection, $f_{w,d}$, between the bottom rail of the diaphragm and the substrate is in withdrawal.

The factor $K_{i,w}$ is the minimum of

$$1.0 \text{ or } [1.0 + (H/\mu L)^2 + (2M_{d,stab}/\mu f_{p,d,t}\, L^2)]^{0.5} + H/\mu L$$

where $M_{d,stab}$ is the net overturning destabilising moment arising from V_H countered by the vertical actions.

Where masonry cladding of the timber frame occurs, some reduction of the wind force on the timber frame is allowed. This is described in Annex D of PD 6693-1 but this shielding effect will be found to be much less than in BS 5268-6.1 and 6.2.

The design racking shear strength of a diaphragm wall, $F_{v,Rd}$, is given by

$$F_{v,Rd} = K_{opening}\, K_{i,w}\, f_{p,d,t}\, L$$

where $K_{opening} = 1 - 1.9\, A / HL$ is the factor to allow for the reduction in racking strength due to the aggregate area A of openings in the diaphragm

$f_{p,d} \quad = F_{f,Rd}\,(1.15 + s) / s$ and $f_{p,d,t} = \Sigma f_{p,}d$ (see 9.7.4.4)

where

$f_{p,d}$ is the design shear capacity of the perimeter sheathing fasteners in kN/m

$F_{f,Rd}$ is the design lateral capacity of an individual fastener in kN

s is the fastener spacing in m.

Where more than one sheathing is used on a diaphragm wall, the total design shear capacity of fasteners is given by $f_{p,d,t} = f_{p,d,1} + K_{comb}\, f_{p,d,2}$

where $f_{p,d,1}$ is the design shear capacity of the perimeter fasteners of the primary sheathing layer and $f_{p,d,2}$ is the design shear capacity of the perimeter fasteners of the secondary sheathing layer

where $f_{p,d,1} \geq f_{p,d,2}$

K_{comb} is the sheathing combination factor, for example

0.75 for secondary sheathing on the opposite side of the framing with the sheathing identical to the primary sheathing with regard to material, thickness, fasteners and fastener spacing

0.50 for secondary sheathing on the opposite side of the framing with the sheathing identical to the primary sheathing with regard to material, thickness, fasteners and fastener spacing

0.50 for secondary sheathing on the opposite side of the framing with the sheathing different from the primary sheathing with regard to material, thickness, fasteners and fastener spacing.

The value of $f_{p,d}$ for wood-based materials may be obtained by calculation in accordance with 8.3 whilst values for various plasterboard thicknesses and combination of thicknesses is given in PD 6693-1. For example, a 12.5 mm plasterboard fixed with 3.2 diameter screws at 300 mm centres has $f_{p,d}$ = 1.27 kN/m and similar sheets fixed on opposite sides of the timber frame has a value of 2.19 kN/m. The typical plasterboard separating wall construction of a minimum 30 mm thickness of plasterboard in at least 2 layers has a value of 2.19 kN/m.

From **Figures 19.22** and **19.23** it can be seen that at the leeward corner there is a vertical reaction. The value of this reaction is given by

$$F_{c,d,leeward} = 0.8\ W_{v,t,d}\ [(M_{d,,dst,\ base} / M_{d,stb}) + 0.6 / L]$$

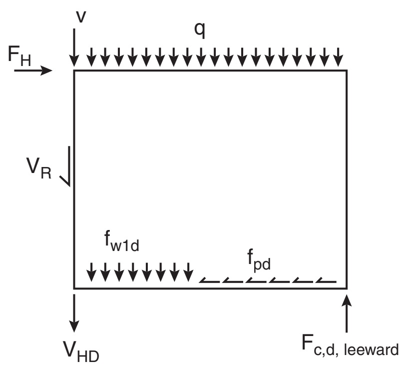


Figure 19.23 Wall type C

where

$W_{v,t,d}$ is the total design vertical load acting on the wall diaphragm in kN

$M_{d,,dst,\ base}$ is the design destabilising moment from design wind load taken about the base of the diaphragm in kNm

$M_{d,stb}$ is the design stabilising moment from the vertical loads taken about the leeward corner of the diaphragm in kNm

L is the length of the diaphragm in m.

Note that to arrive at the maximum value for $F_{c,d,leeward}$ it may be neccessary to consider favourable and unfavourable actions.

Then $F_{c,d,leeward} \leq F_{cR,d}$ where $F_{cR,d}$ is the sum of the design compressive capacities of the studs in kN within 0.1L of the leeward corner of the diaphragm. The design compressive capacity of a stud is the lower of the design axial strength and the design end bearing strength of the stud on the horizontal framing member.

19.8.8 Frames and arches

For plane frames and arches a second order linear analysis should be made of the structure with initial deviations in geometry as shown in **Figure 19.24**.

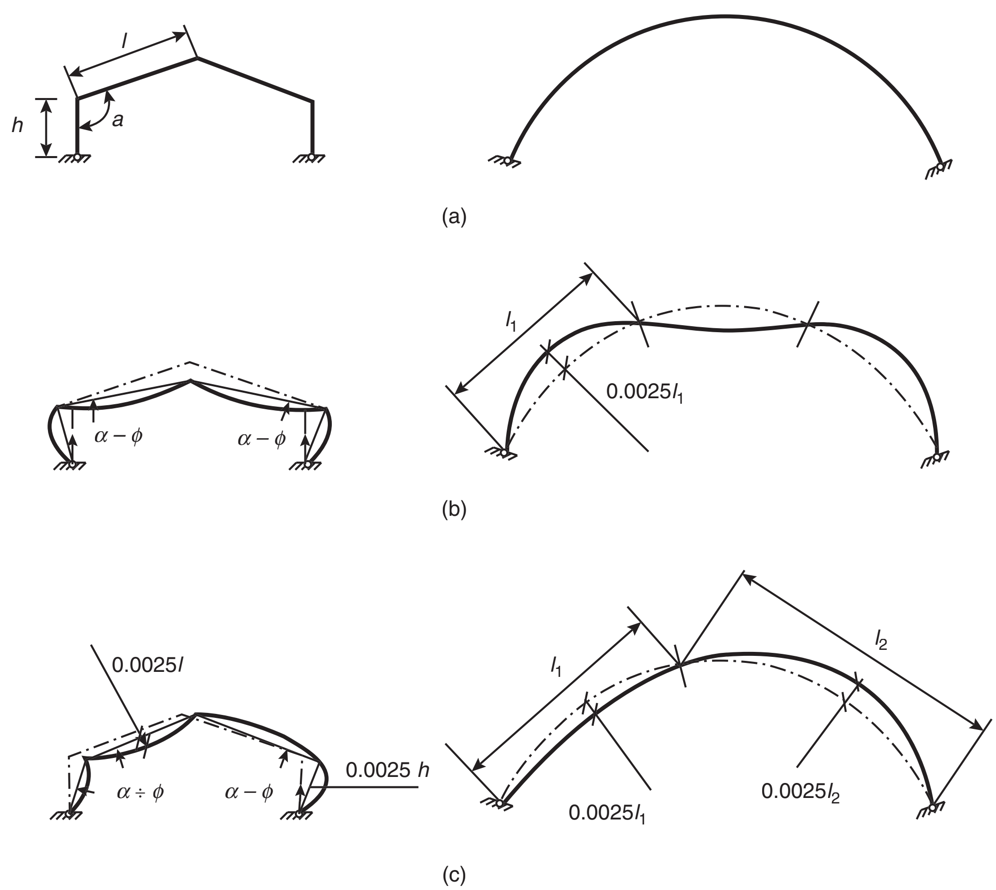


Figure 19.24 Examples of assumed initial deviations (a) for a frame or arch; (b) subject to symmetrical loading; (c) asymmetrical loading (BSI, 2004). Permission to reproduce extracts from BS EN 1995-1-1 is granted by BSI

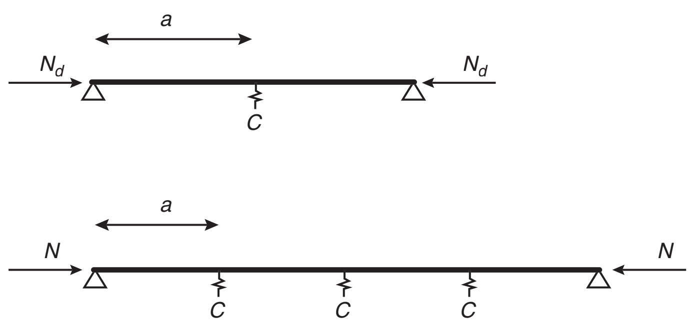


Figure 19.25 Single member braced by lateral supports (BSI, 2004). Permission to reproduce extracts from BS EN 1995-1-1 is granted by BSI

The value of Φ should be at least 0.005 radians for $h \leq 5.0$ m and $0.005\sqrt{5/h}$ for $h > 5.0$m where h is the height in m. There should be an initial sinusoidal curvature between nodes corresponding to an eccentricity of e where e is at least 0.0025l.

The elastic constants to be used in the analysis are

$$E_d = \frac{E_{mean}}{\gamma_M} \text{ and } G_d = \frac{G_{mean}}{\gamma_M} \qquad \text{(see } \textit{EC5-1–1 2.2.2 and 2.4.1(2)P}\text{)}$$

19.8.9 Bracing of members either singly or in a system

EC5-1-1 9.2.5 gives methods of calculating bracing forces for single members and a series of members to prevent instability and excessive deflection.

For single members the forces in the bracing system shown in **Figure 19.24** should be calculated assuming the spring stiffness of the support $C = k_s \frac{N_d}{a}$ where k_s is a factor given in EC5-1-1 **Table 19.2** and has the value 4 in the UK National Annex. The force at the support $F_d = N_d / k_{f,1}$ for a solid timber member and $N_d / k_{f,2}$ for glulam and LVL. The values of $k_{f,1}$ and $k_{f,2}$ are given in EC5-1-1 **Table 19.2** with the values of 60 and 100 respectively specified in the UK National Annex. Note that the difference between solid timber and glulam/LVL is due to the maximum initial bow allowed in the member

The value of N_d for a rectangular beam should be taken as $(1 - k_{crit})\frac{M_d}{h}$ [9.36] where k_{crit} is determined for the unbraced beam, M_d is the maximum design moment and h is the beam depth.

For a series of parallel members which require lateral supports within their span, a bracing system capable of sustaining a uniformly distributed load, q_d in addition to any other horizontal actions such as wind is required as shown in **Figure 19.25**.

The value of q_d is given by $k_L \frac{nN_d}{k_{f,3}L}$ [9.37] where k_L is the minimum of 1.0 and $\sqrt{\frac{15}{L}}$ and $k_{f,3}$ is given in the UK National Annex as 60 where the spacing of members is not greater than 600 mm and 40 otherwise.

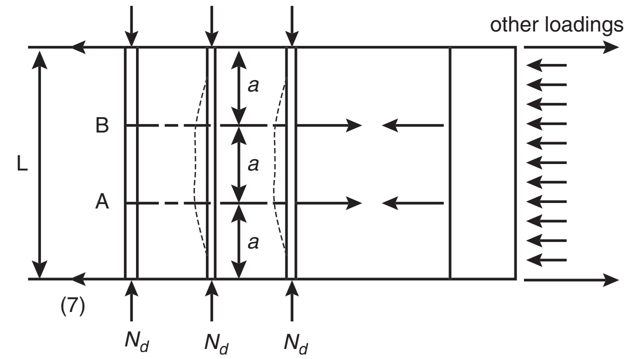


Figure 19.26 Lateral support for a system of members (BSI, 2004). Permission to reproduce extracts from BS EN 1995-1-1 is granted by BSI

The maximum horizontal deflection of the bracing system under q_d and any other actions should not exceed L/500.

19.8.10 Detailing and control

Section 10 of BS EN1995-1-1 describes structural detailing and control of materials, glued joints (but not design), connections and mechanical fastenings. It gives details on nails (e.g. pre-boring), bolts and washers, dowels and screws.

The requirements for transportation and erection are given together with the requirements for quality assurance. Note that a further part of EN 1995 covering the construction of timber buildings is in the course of preparation.

Special rules for diaphragms are given:

(a) for floor and roof diaphragms the maximum spacing of fasteners, nails and screws, around the perimeter of a sheathing sheet should be 150 mm;
(b) for vertical diaphragms the maximum perimeter spacing of fasteners should be 150 mm for nails and 200 mm for screws.

Note that for horizontal diaphragms smooth nails should not be used and whilst not specifically stated with regard to vertical diaphragms it must be assumed that this condition also applies.

Special rules for the dimensional tolerances on fabrication and erection of trussed rafters are given.

19.9 Conclusions

Eurocode 5–1-1 introduces significant changes in the execution of timber design from BS 5268 and its predecessor CP112. Surprisingly much of the technology in EC5-1-1 was known at the time of drafting the first edition of CP112 in 1952 but at that time it was considered slide rules could not cope with the complexity of the consequent calculations. Hence, the simplifications in CP112 many of which were carried forward to BS 5268.

To fully comprehend the effects of time and moisture on the design of timber an understanding of the principles set out elsewhere is required. It may help the reader to solve some those problems that were not described in either CP112 or BS 5268!

The concepts of EC5-1-1 allow consideration of design situations beyond those encountered with normal buildings. It is possible to assess the strength of a structure for durations of load ranging from a fraction of a second to centuries and to integrate new materials into timber design.

19.10 References

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CONTENTS

Chapter 20
Masonry

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This chapter introduces masonry construction and its components. It provides background information on the basic concepts of masonry construction, covering load-bearing and non-load-bearing construction techniques. This chapter also provides details of the masonry units available and widely used in the UK; typical jointing and bonding techniques; and restraint systems often employed in masonry construction. The chapter gives some rules of thumb for initial sizing of various forms of construction to aid with preliminary/initial design. Introductory information is provided on the design of masonry under seismic conditions, and the principles outlined in the Eurocodes. The chapter presents some detailed design examples on masonry under local, vertical and lateral loads.

20.1 Introduction

Masonry is the generic term used to describe the matrix of clay, concrete, stone or calcium silicate units (bricks or blocks) with cement or lime based mortars. Masonry construction is widely used throughout Europe and extensively used in the construction of low to medium rise buildings within the UK.

Masonry is often used in load-bearing construction, cladding to frame buildings or for isolated structures such as free-standing or retaining walls. The choice of construction will often depend on a number of factors, such as:

- Type of space, i.e. residential, office
- Geometry, i.e. clear span or cellular
- Acoustic separation, i.e. acoustic partitioning or floors
- Fire resistance
- Flexibility of spaces
- Aesthetics.

20.2 System selection

20.2.1 Forms of construction

20.2.1.1 Load-bearing masonry

Load-bearing masonry is suited to cellular low rise building forms in modern construction, with much of the domestic construction in Europe being load-bearing masonry. For larger buildings masonry is often specified as a cladding material for aesthetic reasons.

Historically load-bearing masonry has been used for larger building forms (over five storeys). However, these forms of construction are now often unfeasible given the large volume of structure used to create the load-bearing elements.

To utilise load-bearing construction effectively, mitigating complicated detailing or excessive use of ancillary products (wind posts, etc.), buildings should have the following features:

- cellular in plan with consistent cross walls to form stability;
- limited floor to ceiling heights (ideally less than 3 m);
- consistent spacing of walls or avoidance of short walls or piers;
- limitation of large spans onto masonry units;
- avoidance of large openings.

Modern load-bearing masonry construction is often limited to buildings less than three storeys or 12 m. Above this height load-bearing masonry construction can begin to become uneconomical.

20.2.1.2 Cladding

For larger buildings where future flexibility is critical or where large spans required, masonry is often employed as cladding. Cavity construction is the norm, with inner and outer leaves of masonry or an outer leaf of masonry supported by lightweight steel or timber studwork inner leaf.

Masonry used as cladding is still load-bearing as it supports lateral (wind) and vertical (self-weight) loads. This is important, as rules on slenderness and allowable panel size will still apply.

For framed buildings, such as offices, masonry can be constructed in panel sizes in excess of those normal for load-bearing masonry; i.e. 4 m high and 6 m wide. This will result in the need for secondary support elements such as wind posts or head/cill beams.

Masonry cladding is often supported on shelf angles fixed to slab edges. Typically these can sustain 10–14 kN/m, meaning two storeys of masonry. BS 5628 (clause 25.3.2) suggested a limitation of 9 m height or three storeys for support to the outer leaf in these situations.

The detailing of masonry cladding is critical to ensure satisfactory performance. This includes the location of movement joints (horizontal and vertical), restraints, location of support angles and location of secondary supports (wind posts, etc.).

20.2.1.3 Retaining and free-standing walls

Free-standing walls typically need only support self-weight and lateral loads. These are designed as cantilever elements with the flexural strength of the masonry units being used limiting the achievable height or width of the sections.

Geometric sections such as piers, fin walls or diaphragm walls can be employed to increase the allowable height of free-standing walls. These need to be checked as cantilever sections, supporting the masonry panels.

Retaining walls can take many forms and the choice of which can depend on the retained height or finishes required. Examples are:

- Brick or block retaining walls (using piers or return walls).
- Pocket retaining walls (isolated reinforced pockets are used at regular centres to resist bending).
- Grouted retaining walls (shell bedded blockwork is installed over reinforcement, with cavities filled to provide bending resistance).
- Geometric walls.

Movement joints need to be distributed along the length of both free-standing and retaining walls to accommodate thermal movement or potential variations in height or support conditions.

20.2.2 Benefits of masonry

20.2.2.1 Fire resistance

Masonry construction is inherently fire resistant. However, the degree of fire resistance is dependent on the thickness of the wall, the masonry units used and whether the wall is load-bearing or not.

BS EN1996-2 gives information on the fire resistance of masonry. Some typical values are given below (assuming no plaster finishes):

- 100 mm HD brickwork, load-bearing or non-load-bearing single leaf wall, maximum 2 hours' resistance.
- 100 mm solid dense concrete blockwork, load-bearing or non-load-bearing single leaf wall, maximum 2 hours' resistance.

Refer to BS EN1996-2 for further details.

20.2.2.2 Acoustic performance

Acoustic separation must consider airborne and structure-borne sound transmission. Acoustic separation between two spaces can be achieved by mass or physical separation (cavities).

Masonry is used for separating wall construction as most masonry units have inherent mass which can help mitigate airborne sound transmission.

The Building Regulations part E (H.M. Government, 2005) gives limiting values for separating walls for both new and refurbished buildings. For new buildings this specifies a value of between 43 dB and 45 dB (airborne sound) which can be achieved by:

- Solid blockwork walls (min density of 415 kg/m^2 including plaster) with 13 mm thick plastered on either side.
- Solid brickwork walls (min density of 375 kg/m^2 including plaster) with 13 mm thick plastered on either side.
- Cavity blockwork walls (min density of 415 kg/m^2 including plaster) with 50 mm cavity and 13 mm thick plastered on either side.
- Cavity blockwork walls (min density of 300 kg/m^2 including plaster) with 75 mm cavity and 13 mm thick plastered on either side.

Part E gives further guidance on the construction methodology and detailed requirements of these wall types with other parts of the structure to achieve the required performance.

20.2.2.3 Thermal performance

Thermal performance of the fabric of a building is becoming increasingly critical as further guidelines are imposed by European governments.

Manufacturers of bricks and blocks provide specific details on the thermal resistivity of their units which can be used to determine the thermal insulation values of types of wall construction.

Typically masonry walls require some insulation to provide sufficient thermal insulation to meet the UK regulations on thermal performance (Building Regulations part L). Cavity wall construction within the UK allows insulation to be between masonry leaves; for solid walls insulation can be internally or externally placed to improve thermal performance.

20.2.3 Types of masonry units (clause 3.1)

There are several different types of masonry unit available in the UK and Europe. The common UK forms of construction are: clay bricks, aggregate concrete blocks, aerated concrete blocks or manufactured or natural stonework. Other forms of unit are available, but are not covered in this chapter. Primarily the following text will cover clay brickwork and aggregate concrete blocks.

20.2.3.1 Clay brickwork

Clay brickwork is a common form of facade or load-bearing construction unit in the UK. Clay brick units are typically 102 mm × 215 mm long × 65 mm high and are formed using either extrusion or moulding. The mechanical properties are dependent on the clay used, the type of manufacturing and the firing process.

Two types of unit are described in BS EN771-1; these are high density (HD) and low density (LD). Typically HD units are used within the UK and take the form of solid units, units with limited frogs or vertical holes.

Brick units are normally defined by the compressive strength, although in the UK some generic terms are used for specific bandings of compressive strength units. However, other properties are critical when specifying brick units, such as the degree of water absorption and frost and sulphate resistance.

Common forms of clay brickwork are common/facing bricks (7–20 N/mm^2, up to 30%Abs) or engineering bricks

(Class A, >125 N/mm², <4.5%Abs and Class B, >75 N/mm², <7%Abs) (Thomas, 1996).

The design assumptions should ideally be stated within a specification, with reference to manufacturers' specific product details for a unit if known. A specification may include:

- Compressive strength
- Tolerance range
- Durability classification
- Active soluble salts classification
- Water absorption.

20.2.3.2 Aggregate concrete blocks

Concrete blocks are typically 100 mm wide × 440 mm long and 215 mm high, although various widths and lengths of block are available.

Concrete blockwork is commonly used as the internal leaf of masonry buildings or as internal partitions. Given the reduced number of mortar joints, due to the larger unit size, blockwork walling tends to give greater compressive strength when compared to brickwork (Curtin *et al*., 1999).

Aggregate concrete blocks are manufactured in a range compressive strengths; typically 3.5 N, 7 N, 10 N, 15 N and 20 N.

Similar to brickwork, the assumptions made in the design must be stated in a specification. For blockwork the specification may include:

- Compressive strength
- Allowable density
- Thermal conductivity
- Freeze/thaw resistance.

20.2.3.3 Aerated concrete blocks

Aerated concrete blocks are similar to aggregate concrete blocks, yet are formed using fine sand, aluminium powder and pulverised fuel ash (Thomas, 1996).

The units are lightweight meaning that they are easily handled, even with thickness in excess of 215 mm, yet can still achieve compressive strengths of up to 10 N/mm². The low density means that they are often used for internal partitions or the inner leaf of cavity walls.

20.2.3.4 Manufactured stonework

Manufactured stonework is formed in a similar way to aggregate concrete blockwork; yet units resemble natural stone.

Manufactured stonework is defined in BS EN771-5. This code gives testing procedures along with minimum compressive strength values, with the onus on the manufacturer providing the stone properties. The code suggests a minimum characteristic compressive strength of 17.5 N/mm² for a homogeneous unit.

20.2.3.5 Natural stonework

Natural stone is used widely for masonry construction. Natural stone buildings often reflect the stone which available within the local geology. The specification of natural stone is covered within BS EN771-6.

For guidance some approximate lower bound values of stone compressive strengths are given in **Table 20.1**, along with some guidance on the specifics of the type of stone (IStructE, 1996; Thomas, 1996).

20.2.4 Mortar types (clause 3.2)

There are several different types of mortar used for masonry construction; these are normally batched proportions of cement, lime, aggregates (sands) and sometimes additives (plasticisers, pigments, etc.).

Traditional four mixes were presented within BS 5628 (and subsequently the National Annex to BS EN1996), based on proportions of the constituent materials to gain specific compressive strengths. These four mixes gave compressive strengths which were inversely proportional to the ability to tolerate movements; i.e. stiffer stronger mortars are less tolerant to movements and contained greater proportions of cement.

BS EN998-2 gives guidance on the specification of mortar. The new guidance gives the manufacturers of designed mortars a greater range of mortar strengths, as indicated in **Table 20.2**. Mortars to BS EN998-2 are defined as 'M' followed by the compressive strength of the mortar in N/mm².

Unit type	Typical compressive strength (N/mm²⁻)	Water Absorption (%)	Comments
Sandstone	25–40	2.4–10	Can delaminate, sandstones should be laid with the bed face perpendicular to the exposed surface. Sandstone is porous and can stain
Limestone	16–35	1–15	Similar to sandstones, yet typically harder
Granite	140–170	0.19–0.30	Impermeable and well suited to the UK environment, Very good abrasion resistance

Based on lower bound values of natural stone.

Table 20.1 Natural stonework

BS 5628	BS EN 1996	Proportions of constituents from BS 5628-1				Ability to accommodate movement	Increasing compressive strength
		Cement: lime: sand	Cement : sand	Cement (including filler other than lime): sand	Cement (inc lime): sand		
	M20	Designed mortar from Table 1 – BS EN998-2					
	M15	Designed mortar from Table 1 – BS EN998-2					
(i)	M12	1 : 0 to 0.25 : 3	1 : 3	–	–		
	M10	Designed mortar from Table 1 – BS EN998-2					
(ii)	M6	1 : 0.5 : 4 to 4.5	1 : 3 to 4	1 : 2.5 to 3.5	1 :3		
	M5	Designed mortar from Table 1 – BS EN998-2					
(iii)	M4	1 : 1 : 5 to 6	1 : 5 to 6	1 : 4 to 5	1 : 3.5 to 4		
		NB – M4 is the minimum strength for reinforced masonry					
	M2.5	Designed mortar from Table 1 – BS EN998-2					
(iv)	M2	1 : 2 : 8 to 9	1 : 7 to 8	1 : 5.5 to 6.5	1 : 4.5		
	M1	Designed mortar from Table 1 – BS EN998-2					

Further guidance is given in BS EN998-2 Table 1 & NA to BS EN1996-1, Table NA.2 & BS 5628 table 1 (© BSI, London, UK)
Adapted from and courtesy of the British Standards Institution (BSI)

Table 20.2 Typical mortar designations. Permission to reproduce extracts from British Standards is granted by BSI

In accordance with BS EN998-2 mortars can be either designed or prescribed. Designed mixes are performance specified by the designer with the composition defined by the manufacturer. Designed mixes may include the following performance criteria: compressive strength, type of mortar, chloride content, air content, water absorption and maximum aggregate size. A full list is provided in clause 6 and table ZA.1 of BS EN998-2.

Prescribed mixes are similar to those defined in BS 5628 in which mortar is defined using predefined mix proportions. Similar mixes are present in the National Annex to part 1 of BS EN1996, with further guidance given on the permitted types of cement to be used.

Mortars can be manufactured in a number of ways including site mixed, factory mixed or pre-batched.

20.2.5 Bonding of brickwork (clauses 8.1.4)

Brickwork and blockwork can be bonded in a number of ways. Brickwork has greater flexibility to create patterns; however, blockwork can follow similar principles by either cutting of blocks or using special units.

BS EN1996 discusses the bonding of both cut and manufacturer units and suggests that units should generally be bonded together based on experience or proven practice. Typical brickwork bonds are shown in **Figure 20.1** (Chudley and Greeno, 2008).

The basic bond requirements for manufactured brick and blockwork are given in clause 8.1.4.1 as:

- Units < 250 mm high, overlap length at least 40% of the height or 40 mm.
- Units > 250 mm high, overlap length at least 20% of the height or 100 mm.
- At junctions and corners the overlap should be equal to the width of the units.

In natural stonework, bonding requirements in BS EN1996 vary to those of brick or blockwork. Clause 8.1.4.2 suggests minimum overlap of least 40 mm or 0.25 times the dimension of the smallest unit. For natural stonework in cavity construction bonding units with a length approximately two-thirds of the wall thickness should be placed at a maximum of 1 m centres vertically and horizontally.

Stack bonding (masonry with no overlap) can be used for decorative purposes and is normally an architectural feature. BS EN1996 suggests that where masonry bonding does not conform to the above limits, reinforcement should be used to provide adequate resistance, this is often combined with an increased number of wall ties per square metre in cavity construction (Chudley and Greeno, 2008).

20.2.6 Types of construction

20.2.6.1 Cavity walling (clause 8.5.2.2)

Cavity walling has been popular in the UK since the 1930s (IStructE, 1996) and is used in domestic and commercial construction. This form of construction helps mitigate damp by introducing a cavity between the internal and external leaves of masonry. Cavities are now often filled with insulation to increase thermal performance, making it ideal for domestic construction in mild climates.

The internal and external walls are tied together using wall ties at regular centres. Provided the ties have sufficient tensile and

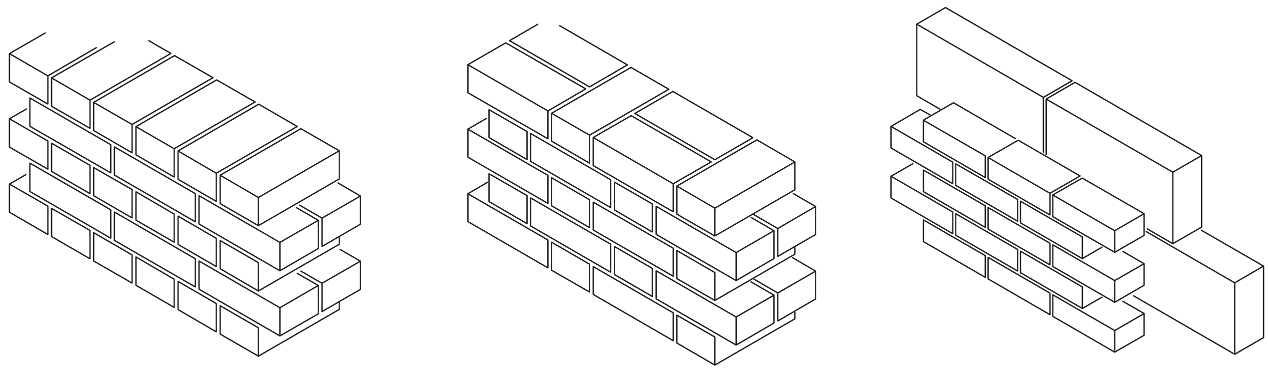


Figure 20.1 Masonry bonding (a) English bond; (b) Flemish bond; (c) Stretcher bond

compressive strength, the tying allows load sharing under lateral (wind) loads and decreases slenderness under vertical loads.

Cavity construction can be recognised by stretcher bonding of the external leaf.

20.2.6.2 Solid walling

Thin solid walls, one brick or block thick, are used for both non-load-bearing and load-bearing partitions. Brickwork partitions are less common in modern construction, although they can be used for aesthetic reasons.

Solid brickwork for external walls of buildings is normally associated with older buildings (i.e. pre-1930s); however, it is widely used in external works such as retaining structures and free-standing walls, where damp penetration would not affect the performance. Where solid brickwork is used, it can be recognised by the brick bonding, i.e. Flemish, English or stretcher (half bonding) (see **Figure 20.1**).

Solid dense or lightweight blockwork walls are prevalent in modern construction. They are used for internal partitions or around core areas and can give greater flexibility as they are manufactured in a range of thicknesses, for example, 70 mm, 100 mm, 140 mm.

Where walls exceed 140 mm wide, these are often constructed using units laid flat, low density units or with specially made units to avoid health and safety concerns over lifting units greater than 20 kg.

20.2.6.3 Collar-jointed construction (clauses 8.5.2.3)

Collar-jointed walls are similar in construction to cavity walls yet have smaller mortar filled cavities, around 20 mm thick. This construction typically uses 100 mm wide units tied together to form 215–220 mm wide construction. This allows solid wall construction using differing masonry units (i.e. brick/block) and mitigates the need for special units or blocks laid flat for larger partitions.

BS EN1996 specifies collar jointed construction should have a minimum of 2 evenly distributed ties per square metre in the UK, calculated in accordance with the clause 6.5 (equation 4). The previous British Standard gave more specific guidance for collar-jointed construction, suggesting these can be designed as cavity walls or solid walls providing the following criteria are met:

- Each leaf is min 90 mm thick.
- Walls are designed based on the strength of the weakest unit and differential movement is considered; i.e. in mixed construction, blockwork will dictate the spacing of joints.
- The eccentricity of the load should be less than 20% of wall thickness.
- 1.5 kN shear ties spaced at 900 mm vertical and 450 mm horizontal centres should be used, embedded a minimum of 50 mm into each leaf. Alternatively bed-joint reinforcement may be used.
- k values in the design of compressive strength are modified.

20.2.6.4 Rubble fill construction

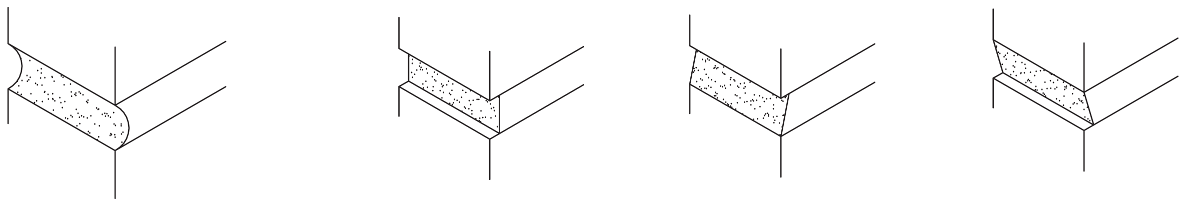
Rubble filled walls are not often employed in modern construction and are not covered by BS EN1996-1. Historically random rubble filled wall construction was used for large masonry walls or piers, where hollow, bonded masonry walls and piers were filled with a mixture of mortar and random rubble (IStructE, 1996).

20.2.6.5 Grouted walls

Grouted walls are sometimes employed for external retaining walls or to improve the strength of cavity walls. These consist of two leaves of masonry separated by a cavity, filled with a grout or concrete. The leaves are tied together with either wall ties or bed-joint reinforcement.

20.2.7 Mortar joints (clause 8.1.5)

Mortar joints are required vertically (perpend-joints) and horizontally (bed-joints) to adequately bond masonry units together. Typically, these are 10 mm thick, allowing dimensional



(a) (b) (c) (d)

Figure 20.2 Masonry jointing (a) Curved/recessed; (b) Square recessed; (c) Struck or weathered; (d) Over struck

coordination, tolerance between the masonry units and for the size of the sand particles used (Thomas, 1996).

BS EN1996 defines limits for general purpose mortar (and lightweight) of 6 to 15 mm thick. Guidance for thin layer joints of 0.5–3 mm is also given, with modification factors to increase the strength of the masonry construction accordingly.

The leading edge of masonry joints may be finished depending on the aesthetic desired. This can be either as work continues (jointing) or by raking out semi-rigid mortar and replacing with a facing or pointing mortar (pointing) (Thomas, 1996). Pointing can allow pigmented mortar to be used, yet reduces the area of load carrying mortar.

Joint finishing can be flush or as shown in **Figure 20.2** (Chudley and Greeno, 2008).

Jointing will often depend on architectural and durability requirements. Generally tooled joints (struck or recessed) can be prone to frost damage and are less water resistant (IStructE, 2008).

Repointing can be used to improve durability of old masonry, where joints have recessed due to weathering. Careful consideration is required for repointing to ensure compatibility of the repointing mortar with the existing masonry units and mortar. Repointing mortar must be suited to the environment and should not be stronger than the existing masonry units or mortar (BRE 1994).

20.2.8 Ancillary products

20.2.8.1 Wall ties (and frame cramps, etc.)

Wall ties are used to join two leaves of masonry together, join masonry to other materials (frame cramps) or to start masonry walls from existing masonry construction (starter ties).

For cavity wall construction, ties transfer the wind loads and stability forces between leaves such that the lateral resistance and the slenderness can be based on the two leaves acting together.

There are many manufacturers of wall ties and each provides data for the compressive, tensile and shear resistance of particular types of tie. Typically the compressive strength of wall ties is critical as they can be slender. BS EN845-3 gives guidance for the compressive strength and suggests that the manufacturer of the wall ties provides declared compressive strengths. Ties were classified in the UK within BS 5628-1 as types 1 to 4. This code of practice gave the type of tie in relation to geographical location, suitable type of construction, density of ties and compressive and tensile resistance. UK manufactures often relate their products to these code of practice references. Some examples are given in **Table 20.3**.

BS EN1996, clause 6.5, gives guidance on the number of wall ties per square metre as equal to the factored wind load per unit area divided by the declared strength of the wall ties. The declared strength is the characteristic (published) strength divided by the factor of safety, $_m$, for ancillary products.

Standard practice from the UK is defined in the previous British Standards and generally equates to 2.5 to 3.4 ties per sq m with an increased number of ties local to door reveals and free edges, as indicated in **Figure 20.3**.

Frame cramps is a generic term for ties used to tie masonry to other elements, for example, steel, timber or concrete frames. Manufacturers give declared strengths which are used to determine the number of ties to provide 'simple' support to a panel. It is important to ensure an adequate load path if such

	Type of structure	Compressive resistance (M2 mortar)	Tensile resistance (M2 mortar)
Type 1	Heavy Duty Ties – Suitable for most types of construction. Not very flexible, therefore not suitable for applications where large differential movements may occur, i.e. masonry cladding to timber frames	2500	2500
Type 2	General Purpose Ties – Suitable for small domestic or commercial buildings up to 15 m high	1300	1800

Further guidance is given in Table C1 and C3 from BS 5628-1 (© BSI, London, UK)
Adapted from and courtesy of the British Standards Institution (BSI)
*Values for 1 mm displacement

Table 20.3 Typical wall tie properties. Permission to reproduce extracts from British Standards is granted by BSI

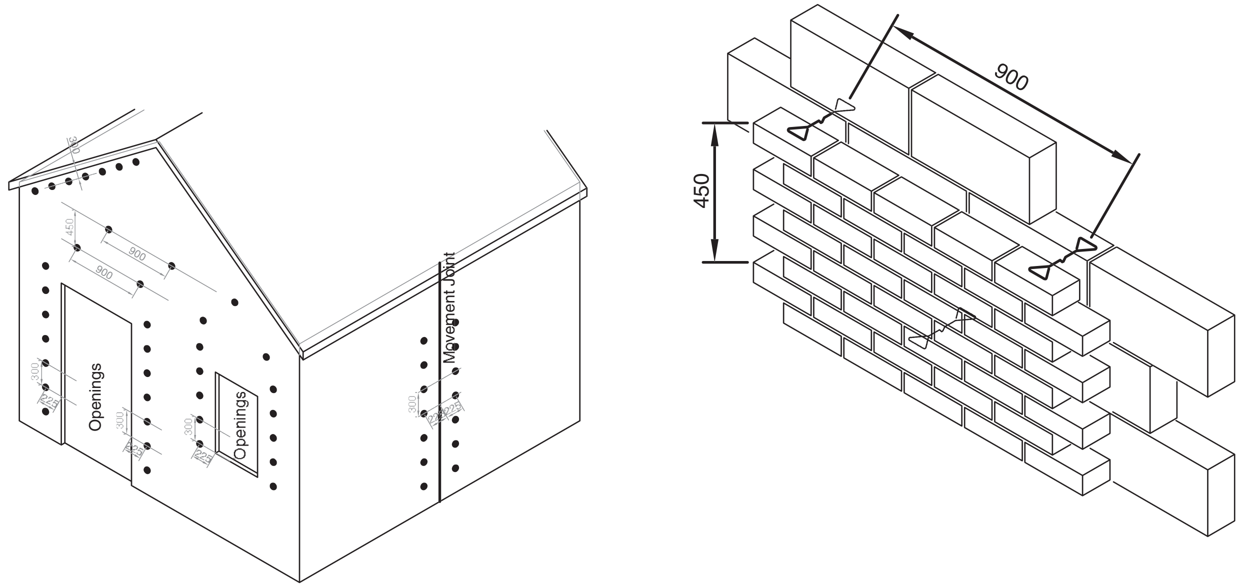


Figure 20.3 Location of wall ties in typical construction

ties are used; i.e. that the supporting structure is designed for the lateral loads imposed by the restrained wall panel.

20.2.8.2 Restraint straps and joist hangers

Restraint straps tie masonry to other forms of construction to ensure adequate restraint. Typically, these are used to restrain wall panels laterally where floors span parallel to walls, restrain wall panels laterally where floors span perpendicular to walls where the bearing along cannot provide restraint and where roofs require vertical restraint against uplift loads. Examples of such strapping details are indicated in **Figure 20.4**.

Joist hangers of certain types can be used to support timber joists, yet also provide restraint to wall panels. Typically the joist hangers must have a return to the back of the restrained wall of 75 mm (IStructE, 2008).

Designers should consider how walls are tied into the floor or roof structures if support or restraint is assumed in design. Details or specifications should then reflect this.

20.2.8.3 Masonry supports (framed construction)

Shelf angles are fixed to slab edges and will support up to three storeys of brickwork. The brickwork must be laterally restrained by either a lightweight cold rolled inner leaf framing system or a blockwork infill panel.

Horizontal movement joints are required to the underside of these shelf angles to allow for tolerance and movements of the supporting structure. Proprietary systems are available and are normally designed by the manufacturers; however, understanding the principles when detailing masonry for cladding to framed buildings is important (see **Figure 20.5**).

20.2.8.4 Wind posts

Wind posts support the leading edges of masonry panels where additional restraint is required. Wind posts can be proprietary 'off the shelf' items based on manufacturers' details or can be designed using structural steelwork sections. Masonry panels are tied to wind posts using standard frame cramps or lengths of steel acting as flat wall ties.

Many ancillary products are stainless steel; however, lower specifications can be used depending on location. BS EN1996 part 2 gives guidance on the selection of materials for specific locations.

20.2.9 Durability of masonry

The durability of masonry depends on the suitability of the material for the location. The specification of the masonry units, mortar and ancillary products is paramount to ensure durable construction.

BS EN1996 part 2 considers the selection of materials for different types of use and location. Part 2 suggests the consideration of macro (temperature variations, humidity, snow, wind and rain) and micro conditions of the site.

The micro exposure conditions of the site are given five categories MX1 to MX5:

- MX1 Dry environment.
- MX2 Exposed to moisture or wetting.
- MX3 MX2 + possible freeze/thaw.
- MX4 Exposed to saturated salt air or seawater.
- MX5 Exposed to aggressive chemicals (e.g. in ground or ground water sulphates).

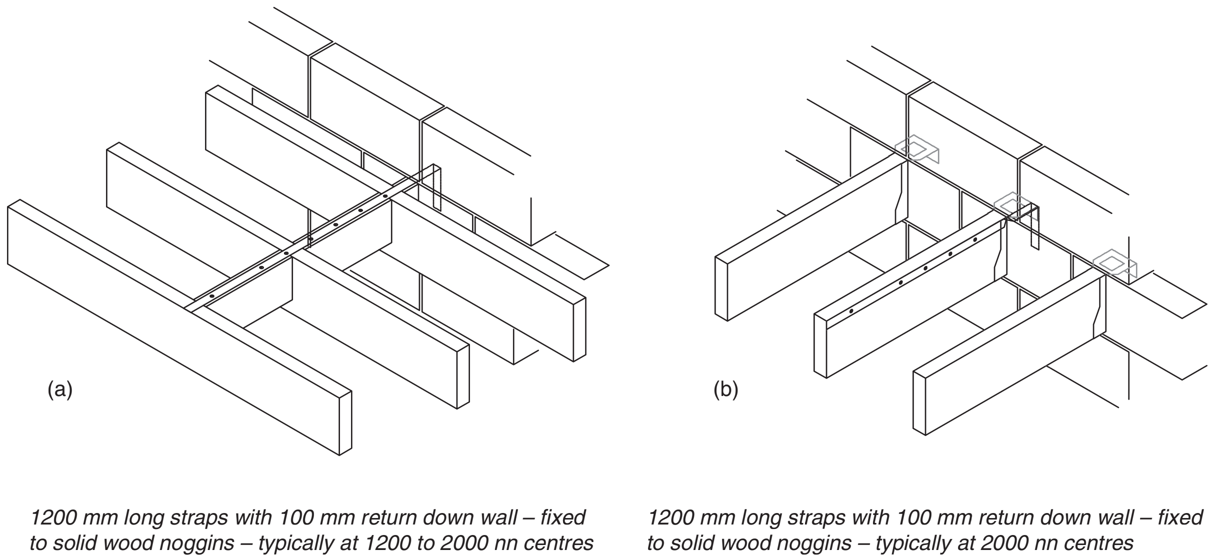


Figure 20.4 Restraint straps (a) Timber joists parallel; (b) Timber joists perpendicular

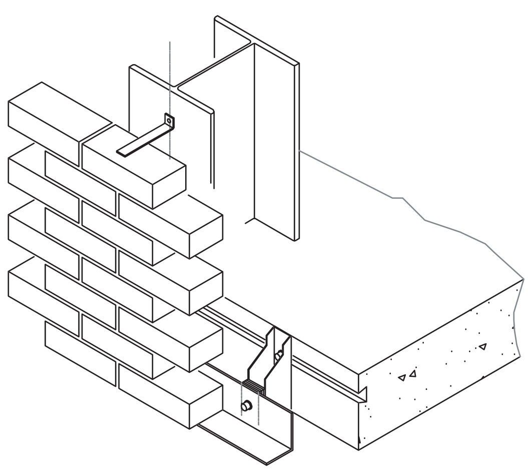


Figure 20.5 Masonry cladding on support angles

Masonry units, mortar and ancillary products should be specified based on the assumed exposure conditions. **Table 20.4** and **Figure 20.6** relate exposure conditions to masonry unit type and mortar which should be specified. **Table 20.4** has been modified to account for the guidance in BS 5628-3 as the National Annex (NA) to BS EN1996 precludes the use of annexes B and C of BS EN1996-2.

F0, F1, F2 — Freeze/thaw resistivity of masonry unit specification. Defined as Passive (F0), Moderate (F1) or severe (F2) (refer to BS EN771-1). If passive exposure conditions are used mortars need to be protected against freeze/thaw during construction.

S1, S2 — Active soluble salt content category (BS EN771-1) gives guidance on limits on total percentage of soluble salts by mass. When S1 units are specified for conditions MX2.2, MX3.2, MX4 and MX5 sulphate resisting mortars should be used.

P, M, S — Passive (P), Moderate (M) or Severe (S) exposure condition for mortar designation (refer to BS EN998-2). If passive exposure conditions are used mortars need to be protected against freeze/thaw during construction.

Ancillary products are typically stainless steel, galvanised or plastic. BS EN1996-2 Tables C.1, C.2 and C.3 give guidance on the material specification for ties, straps, lintels and bed-joint reinforcement in relation to categories MX1 to MX5.

Austenitic stainless steel components (ties, straps and lintels) have unrestricted use for categories MX1 to MX4, with a requirement to consult the manufacturers for category MX5. Bed-joint reinforcement can be used unrestricted for categories MX1 to MX3.

Galvanised components generally have unrestricted use for category MX1. However, certain elements can be used in more severe categories (MX2 and MX3) depending on the g/sqm of coating and the type of component.

20.3 Preliminary sizing

20.3.1 Overall stability

Structural stability of masonry buildings can be provided in a number of ways depending on the form of building. In all forms the stability system should resist loads in two orthogonal directions, as well as any twisting forces due to asymmetry of applied loading or building geometry.

Exposure class			Clay masonry Units (HD only)	Aggregate concrete blocks	Mortar specifications
MX1	Dry environment	Inner leaf of cavity walls, internal partitions or masonry isolated from adjacent damp masonry Assuming that the masonry is not exposed to severe conditions during construction	High Density (HD) F0, F1 or F2 and S0, S1 or S2	Any	P, M or S (M12, M6, M4 or M2)
		Rendered external walls protected from severe wetting	High Density (HD) F1 or F2 and S1 or S2	Any	P, M or S (M12, M6 or M4)
MX2.1	Exposed to moisture or wetting	Internal masonry exposed to moisture (e.g. plant rooms), sheltered external masonry not subject to severe driving rain or frost	High Density (HD) F0, F1 or F2 and S0, S1 or S2	>7.3 N/mm^2 with min density 1500 kg/m^3	M or S (M12, M6 or M4)
		Masonry below ground in well drained non-aggressive soils	High Density (HD) F0, F1 or F2 and S0, S1 or S2	>7.3 N/mm^2 with min density 1500 kg/m^3	M or S (M12, M6 or M4)
		Un-rendered external walls	High Density (HD) F1 or F2 and S1 or S2	Any	M or S (M12, M6 or M4)
		Masonry in parapets not exposed to frost or aggressive chemicals	High Density (HD) F1 or F2 and S1 or S2	>7.3 N/mm^2 with min density 1500 kg/m^3	M or S (M12, M6 or M4)
MX2.2	Exposed to moisture or severe wetting	Masonry below ground in well drained non-aggressive soils with severe wetting	High Density (HD) F1 or F2 and S1 or S2	>7.3 N/mm^2 with min density 1500 kg/m^3	M or S (M12 or M6)
		Un-rendered external walls	High Density (HD) F2 and S1 or S2	Any	M or S (M12, M6 or M4)
		Masonry in parapets not exposed to frost or aggressive chemicals, yet with severe wetting	High Density (HD) F2 and S1 or S2	>7.3 N/mm^2 with min density 1500 kg/m^3	M or S (M12 or M6)
MX3.1	Exposed to moisture or wetting with potential for freeze/thaw	Masonry below ground in well drained non-aggressive soils with potential for freeze/thaw	High Density (HD) F1 or F2 and S1 or S2	>7.3 N/mm^2 with min density 1500 kg/m^3 Freeze/thaw resistant	M or S (M12 or M6)
		Masonry in parapets not exposed to frost or aggressive chemicals with potential for freeze/thaw			
MX3.2	Exposed to moisture or severe wetting with potential for freeze/thaw	Masonry below ground in well drained non-aggressive soils with severe wetting with potential for freeze/thaw	High Density (HD) F2 and S1 or S2	>7.3 N/mm^2 with min density 1500 kg/m^3 Freeze/thaw resistant	S (M12 or M6)
		Masonry in parapets not exposed to frost or aggressive chemicals, yet with severe wetting with potential for freeze/thaw			
MX4	Exposed to saturated salt air or seawater	Masonry in coastal areas or adjacent to roads which may be subject to de-icing salts	Consult manufacturer		
MX5	Exposed to aggressive chemicals	Masonry in contact with sulphate containing soils or groundwater Masonry in contact with highly acidic soils, contaminated ground or groundwater Masonry near airborne aggressive chemicals	Consult manufacturer		

BS EN1996-2 – Tables A.1, B.1 or B.2 & BS 5628-3 Table 12 (© BSI, London, UK)
Adapted from and courtesy of the British Standards Institution (BSI)

Table 20.4 Exposure class / masonry properties. Permission to reproduce extracts from British Standards is granted by BSI

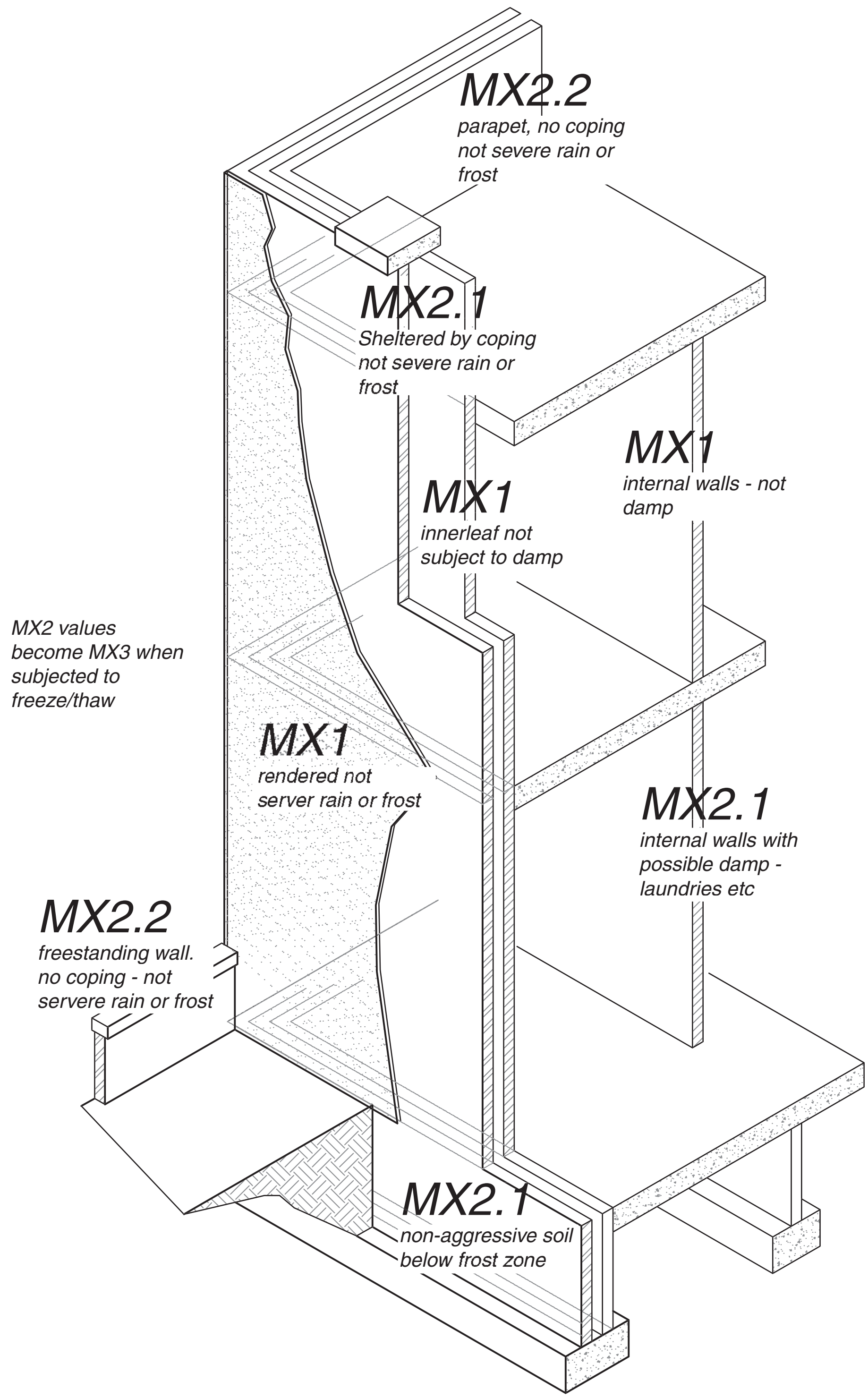


Figure 20.6 Exposure classifications and masonry properties

In load-bearing masonry construction stability is provided by either a cellular plan, cross wall construction, spine wall construction or by using geometric sections (e.g. diaphragm or fin walls) (IStructE, 2008) (see **Figure 20.7**).

Cellular plan construction is often found in low to medium rise construction where the internal and external walls form stiff cellular boxes which can be used to resist lateral loads. The internal and external walls in this form of construction

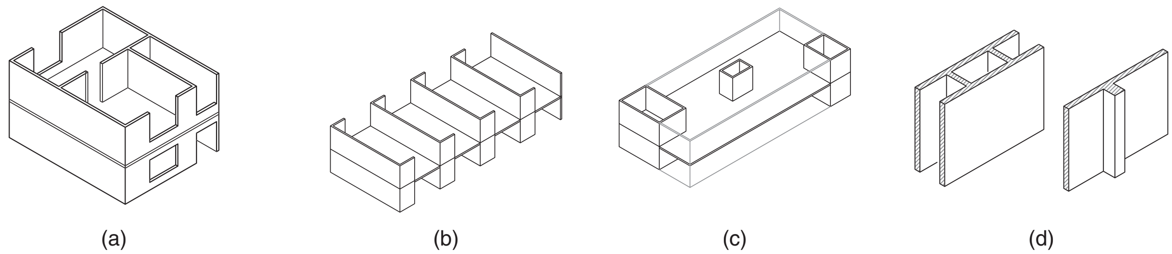


Figure 20.7 Building type (a) Cellular; (b) Cross wall; (c) Spine wall; (d) Geometric sections

tend to be load-bearing and provide the vertical and lateral resistance.

Cross wall construction is suited for long narrow building forms, such as terraced housing, where strong lines of continuous construction are spaced at regular centres.

Spine wall construction utilises stiff masonry elements located at key locations throughout a building. These may be at lift or stair cores, gable walls or specific internal walls which are continuous to foundation level.

Geometrical sections use stiff section shapes to created stable structural elements, such as fin and diaphragm walls.

In all forms of construction lateral loads are resisted by a mixture of the flexural strength and compressive resistance to resist lateral loads.

All forms of stability rely on the basic equation:

$$\frac{f_x}{\gamma_m} + \frac{P}{A} > \frac{M}{Z}$$

In the equation calculated lateral loads are used to determine overturning moments (M), these are divided by the elastic modulus of the section shape (Z) to determine a bending stress. This bending stress is checked against the axial stress and flexural strength.

To achieve stability often sufficient vertical load is required to overcome the overturning forces; this should be considered when determining the span of floor or roof elements as lightly loaded walls may not offer much lateral resistance.

To transfer loads from the facades to the chosen lateral force resisting system, floors and roofs must act as stiff diaphragms. Diaphragm action can normally be achieved using, for example, timber floors with sufficient noggins and decking, using precast concrete floors with structural toppings or timber roof construction with sufficient bracing. In all cases, the tying and restraint details are critical to transfer loads from the facade to the diaphragm and the stability elements – as defined by clause 8.5.1.1 note 2.

For non-load-bearing masonry cladding to frame construction, stability is normally provided by a separate system, for example, concrete cores, steel bracing or sway frame action. When checking the stability system in framed construction it is important to limit the inter-storey drifts to ensure the masonry cladding does not crack, for example, steel framed buildings often consider limiting deflections to between h/300 to h/500.

20.3.2 Local stability

In non-load-bearing and load-bearing construction masonry panels are checked for local stability under vertical and lateral loads. Local stability is a function of the support conditions, height, length and thickness of the section.

Under vertical loads the compressive strength of masonry is proportional to the slenderness of the section, similar to other materials. The connection of a masonry panel to adjacent structure, i.e. floors, walls or roof will determine the 'effective' height of a section.

Under lateral loads support conditions determine the applied loads within the panel for both load-bearing and non-load-bearing construction (based on yield line analysis). Local lateral loads are typically higher than those considered for overall stability, as overall loads include reduction factors for the non-simultaneous action of wind on faces of a building (BSI, 2002), making local element checks critical in some cases.

The following gives some notes on assumed levels of restraint for wall panels when designing for vertical and lateral loads. The exact details should be reviewed depending on project specific data.

20.3.2.1 Top and bottom supports

For vertical loads the effective height of a masonry wall is defined as follows (BS EN1996-1 clause 5.5.1.2):

$$h_{ef} = \rho_n h$$

where h is the height of the wall and ρ_n represents a number of reduction factors which may be applied to account for the restraint conditions.

Three examples of restraint offered by the connection of adjacent floors are given in BS EN1996, with corresponding values of ρ_2.

$$\rho_2 = 1.00$$

'Simple resistance' means that the wall is designed as the actual distance between lateral supports (floors) and can be achieved in a number of ways:

- the wall has reinforced concrete floors or roofs spanning at the same level on both sides, yet has an eccentricity of load of greater than 25% of the thickness of the wall, or
- when the wall has reinforced concrete floors or roofs on one side with a minimum bearing of at least 2/3 the thickness of the wall; yet has an eccentricity of load of greater than 25% of the thickness of the wall, or
- for walls restrained by timber floors which span from both sides at the same level, or
- for walls with timber floors on one side which have a bearing of ⅔ the thickness of the wall, where the wall is a minimum of 85 mm thick.

$$\rho_2 = 0.75$$

'Enhanced resistance' can be achieved when either:

- the wall has reinforced concrete floors or roofs span at the same level on both sides, or
- when the wall has reinforced concrete floors or roofs on one side with a minimum bearing of at least ⅔ the thickness of the wall (previously in BS 5628 this was ½ the thickness or 90 mm).

Clause 8.5.1 defines the connection requirements for the tying of walls under lateral loads. Generally simple support is assumed at the head of walls or continuity may be difficult to achieve (IStructE, 2008). In some instances, masonry walls may have no lateral support at the head and should be considered as a free edge.

Simple support for lateral loads can be assumed under the following conditions:

- By providing metal restraint straps capable of transmitting compressive and tensile forces between the wall and floor diaphragm, spaced at 2 m horizontal centres in buildings less than four storeys (BS EN1996-1 – clause 8.5.1.2).
- By frictional resistance between concrete floors or roofs (BS EN1996-1 – clause 8.5.1.3).
- Where damp proof courses are required and can transfer shear (IStructE, 2008).
- Where shear ties are connection to the slab or floor above (IStructE, 2008).

'Enhanced resistance' or continuous supports can be achieved:

- At the base of walls where sufficient vertical load is present to give some moment of resistance.
- Where walls are continuous past supports (i.e. external leaves of cavity walls).

20.3.2.2 Side and edge supports

Within BS EN1996 stiffening effects of adjoining masonry can be considered to contribute to reducing the effective height of the wall under vertical loads (clause 5.5.1.2). Walls are considered to be stiffened by adjacent walls provided certain criteria are met, for example:

- Cracking between the stiffened and stiffening wall is not expected.
- The connection between the stiffened and stiffening wall can transfer tension and compression.
- The thickness of the stiffening wall is at least 30% of the stiffened wall and the length is 1/5 of the clear height.
- The length of the stiffened wall is less than 30 t for two stiffening walls or 15 t for one stiffening wall, where t is the wall thickness

When considering lateral loading on wall panels, the support conditions can be critical. Masonry is anisotropic and is generally stiffer horizontally; therefore the edge supports can govern the capacity of the panel.

Figure 20.8 gives some guidance on the requirements to achieve simple or enhanced restraint for wall panels based on similar guidance given in BS 5628-3:2005 (BSI, 2005). Typically enhanced support can only be achieved when there is sufficient masonry returns bonded into the supported wall. Return or stiffening walls in this situation are ideally equal in thickness to the supported wall and have a return dimension of 10 times that thickness.

Return walls, piers or columns need to be checked to ensure that the supporting structure can sustain the loads applied from the supported masonry.

20.3.3 Rules of thumb and useful information

20.3.3.1 Overall stability

From BS EN1996 part 1-1, the following equation is given to check that sufficient stability elements are provided to prevent sway:

Total height × (design vertical load at foundation level / sum of the stiffness of all stiff walls in direction considered)$^{1/2}$

< 6 for buildings greater than 4 storeys,

< 0.2 + 0.1x number of storeys for buildings between 1 and 4 storeys.

Generally, if this is not satisfied additional walls are required.

20.3.3.2 Vertical loading elements

For vertical loads simple span/depth ratios can be used to preliminary size elements (Adler, 2000).

Masonry columns	max height 4 m	height/min thickness = 15–20 Walls fail in buckling (slenderness) when h/t>10
Masonry walls (single)	max height 5 m	height/min thickness = 20 Walls fail in buckling (slenderness) when h/t>10
Masonry walls (cavity)	max height 5 m	height / min effective thickness = 20 Effective thickness = 2/3 (combined thickness of walls).

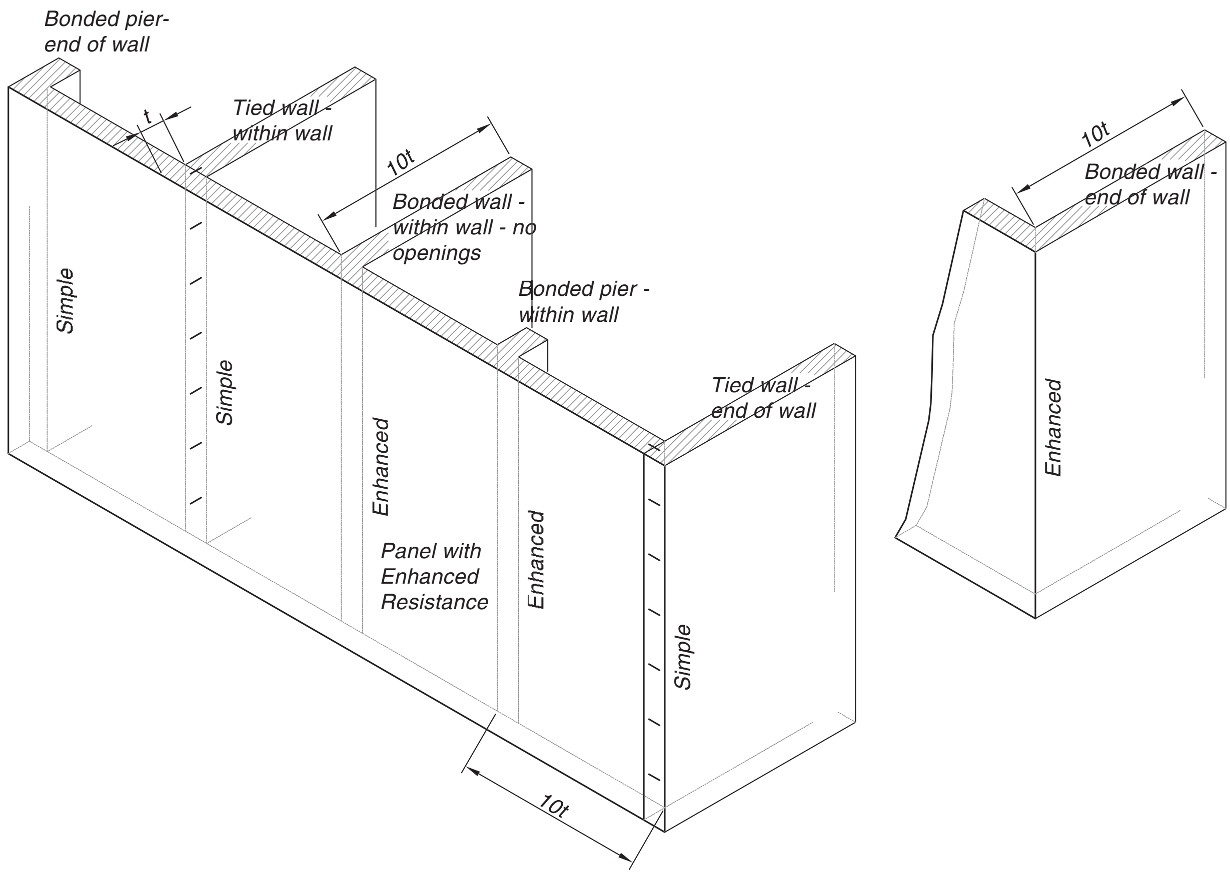


Figure 20.8 Lateral support conditions

The UK Building Regulations (H.M. Government, 2000) and the British Standard for low rise buildings (BSI, 2005) give some guidance on the minimum thickness of walls under specific conditions for low rise buildings, such as:

Internal load-bearing walls min 140 mm block or 215 mm brick where supporting two upper storeys.

Internal non-load-bearing walls min 75 mm and restrained at head.

20.3.3.3 Laterally loaded elements

Laterally loaded panels will span vertically and horizontally between supports depending on the aspect ratio of the panel and the support conditions. As such it is difficult to provide simple span to depth ratios for masonry panels under lateral loads.

However, the following should give some guidance for typical panels:

Type A – simply supported on 3 sides, free top edge	Brickwork panel	L / t = 23,
	Blockwork panel	L / t = 17
Type E – simply supported on 4 sides	Brickwork panel	L / t = 28
	Blockwork panel	L / t = 20

The above based on: wind load = 1 kPA, height = 2.8 m, brickwork abs 7–12% and 7N blockwork.

20.3.3.4 Parapets

Simple rules for parapets are given in the UK Building Regulations (H.M. Government, 2000) and the British Standard for low rise buildings (BSI, 2005). These are summarised as follows:

Solid walls	height 0.6–0.86 m	height/thickness = 4
Cavity walls	height 0.6–0.86 m	height/thickness = 3–4
		Based on combined thickness of wall

20.3.3.5 Retaining and free-standing walls

The BRE have produced a good level of guidance for preliminary sizing of retaining walls (BRE, 1994) and free-standing walls (BRE, 1994) under certain conditions.

For simple retaining walls BRE GBG27 gives the following guidance:

Brickwork walls	height 0.8–1.7 m	height/thickness = 3–4
Blockwork walls	height 0.8–1.7 m	height/thickness = 2.5–3.5

The height/thickness ratio can be increased where piers are introduced or reinforced cavities are used. Refer to BRE GBG 14 for specific limitations/assumptions.

Brickwork walls height 0.5–2.2 m height/thickness = 4.5–6.5

Blockwork walls height 0.4–1.8 m height/thickness = 4–6

The values above are based on moderately exposed locations. Refer to the BRE GBG for specific limitations/assumptions.

20.4 Seismic design

Buildings within seismic or earthquake zones need to be designed with suitable resistance to the horizontal forces generated and/or a suitable level of detailing to accommodate the potentially large movements. The design of structures within seismic zones is discussed in Eurocode 8 (BS EN1998-1:2004) and the National Annex to this code (NA to BS EN1998).

Seismic forces are primarily horizontal (lateral) and must be resisted by a building's lateral stability system. In most cases seismic loads will exceed other lateral loads, and are normally a critical load case. In simplistic terms, seismic forces applied to a structure are proportional to the stiffness and mass of the structure, the ground/site conditions, the building geometry and the type/use of the building. The seismic forces determined by the site and ground conditions are modified by a factors related to the building stiffness and an importance factor which defines a buildings use/risk should it fail.

Masonry structures are used in three forms of construction: unreinforced, reinforced and confined masonry structures, with unreinforced and reinforced masonry structures covered in clause 9 of Eurocode 8.

Unreinforced masonry can be problematic under seismic loads given the lack of ductility of this form of construction. The ability of unreinforced masonry to sustain lateral seismic loads depends on the strength of the units, mortar and the quality of construction. Typically unreinforced masonry buildings can only be used in low seismicity zones with building heights limited to less than four storeys (Chalson, 2008).

Reinforced masonry construction has greater ductility than unreinforced masonry buildings and therefore can be used for taller buildings, up to six storeys (Chalson, 2008).

Confined masonry is the description given to masonry panels located within beam and column frames, such as steel or concrete frames. In this form of construction, masonry panels act as stiffening elements to frames. The masonry acts as diagonal struts within each frame to sustain lateral loads in compression/crushing.

In a load-bearing masonry frame, shear walls often form the lateral force resisting system. For this form of construction clause 9 gives the following minimum values to be used in the design:

- Masonry strength (fb) = 5 N/mm^2 (or 2 N/mm^2 parallel to bed face)
- Mortar strength (fm) = 5 N/mm^2 (or 10 N/mm^2 for reinforced)

The clause also gives guidance on the design methods for masonry buildings including:

- Minimum wall lengths and thicknesses (table 9.2).
- Permitted material factors of safety under seismic (typically 2/3 of the normal values – clause 9.6).
- Analysis methods (typically assuming moment resisting frames with the bending and shear stiffness of elements taken as 50% of the actual values).
- Construction requirements for masonry to floor diaphragm junctions.

Clause 9.7 gives simple rules for the design and construction of buildings which have an importance category I or II (clause 9.7 and table 4.3). The Simple Building clauses would apply to ordinary buildings (II), for which the risk of collapse would not have dramatic consequences, and for low risk buildings (I).

These simple rules can be used to determine the percentage of area required to be shear wall, based on the floor area, storey height and site acceleration. This information is presented in table 9.3, which indicates that unreinforced masonry can really only be used at low site accelerations and for less than three storeys.

The clauses also state that shear walls should:

- Carry 75% of the vertical load.
- Be a minimum of 30% of the length of the building in the particular orthogonal direction.
- Be spaced at least 75% of the building width apart.
- Buildings should be regular in shape (i.e. rectangular with limited recesses or projections).

For non-load-bearing masonry in framed construction, special consideration is required for the detailing of the junctions between the masonry panel and the framed construction. As seismic loading and deflections are dynamic and are often large, masonry infills to structures often need large movement joints to the sides and head. This will often mean complete separation of the frame and the panel of in excess of 50 mm per storey.

20.5 Final design

20.5.1 The Eurocode system

BS EN1996 deals with the structural design of masonry. This Eurocode is split into four parts, which are as follows:

Part 1-1	General – Rules for reinforced and unreinforced masonry structures
Part 1-2	General – Structural fire design
Part 2-0	Design considerations, selection of materials and execution of masonry
Part 3-0	Simplified calculation methods for unreinforced masonry structures.

A series of National Annexes have been issued to accompany the Eurocode. These documents give country-specific data alternative values or clauses specific to the UK.

20.5.2 Basics

BS EN1996 is a limit state design code, as such forces and materials are modified by factors of safety. Within BS EN1996 various factors of safety are given to be applied to loads, masonry units and ancillary materials.

20.5.2.1 Factors of safety (clause 2.4.3)

Factors of safety to be applied to loading have been covered earlier within this book (refer to Chapter 10: *Loading*). The material factors of safety within BS EN1996 are dependent on the materials used, quality of construction and type of loading.

Masonry units are either category I or II, depending on the manufacturing process. Category I units have a declared compressive strength and is normally specified by the manufacturer.

The UK National Annex gives guidance for two classes of construction: class 1 and class 2:

Class 1 – workmanship is undertaken to BS EN1996-1-1, with additional measures relating to supervision and mortar specification.

Class 2 – workmanship is undertaken to BS EN1996-1-1.

Table 20.5 gives the material factors of safety. Additional factors of safety are provided for ancillary products such as reinforcement steel, lintels, wall ties and straps (refer to table NA.1 of the UK National Annex to BS EN1996).

	Class 1	Class 2
Category I Units		
– in direct or flexural compression	2.30[a]	2.70[a]
– reinforced and in direct or flexural compression	2.00[a]	N/A class 1 only
– in flexural tension	2.30[a]	2.70[a]
– in shear	2.50[a]	2.50[a]
– reinforced and in shear	2.00[a]	N/A class 1 only
Category II Units		
– in direct or flexural compression	2.60[a]	3.00[a]
– reinforced and in direct or flexural compression	2.30[a]	N/A class 1 only
– in flexural tension	2.30[a]	2.70[a]
– in shear	2.50[a]	2.50[a]
– reinforced and in shear	2.00[a]	N/A class 1 only

Further guidance is given in NA to BS EN1996-1-1:2005 – Table NA.1 (© BSI, London, UK)
Adapted from and courtesy of the British Standards Institution (BSI)
[a] values can be halved when considered under accidental load cases.

Table 20.5 Masonry material factors of safety. Permission to reproduce extracts from British Standards is granted by BSI

	Coefficients for general purpose mortar
Clay units	
Group 1	k = 0.50, α = 0.70, β = 0.30
Group 2	k = 0.40, α = 0.70, β = 0.30
Aggregate concrete blocks (values for Aerated blocks are similar)	
Group 1	k = 0.55 (0.50[a]), α = 0.70, β = 0.30
Group 2	k = 0.52, α = 0.70, β = 0.30

Further guidance is given in NA to BS EN1996-1-1:2005 – Table NA.4 (© BSI, London, UK)
Adapted from and courtesy of the British Standards Institution (BSI)
[a] for blocks laid flat

Table 20.6 Masonry compressive strength factors. Permission to reproduce extracts from British Standards is granted by BSI

20.5.2.2 Compressive strength (clause 3.6.1.2)

Masonry is a composite material which consists of masonry units and mortar. The compressive strength of masonry is derived as a function of the compressive strength of the units and the strength of the mortar.

$$f_k = k \times fb^{\alpha} \times f_m^{\beta}$$

Fb is the normalised compressive strength of the units and fm is the compressive strength of the mortar. α,β and k are constants applied to the compressive strengths which are dependent on the mortar type, density and thickness and also the type of masonry unit (group 1 and 2). Values of α and β are modified for thin layer or lightweight mortars.

Values for clay units and aggregate concrete blocks in general purpose mortar are given in **Table 20.6** (extract from National Annex to Eurocode 6 – Table NA.4).

The National Annex gives further guidance for specific situations when calculating compressive strengths of various construction types:

- Mortar strength, fm, is to be limited to twice the compressive strength ($2f_b$) of the masonry unit or M12.
- For collar jointed walls the value of k should be multiplied by 0.8.
- Masonry unit strength, fb, is to be limited to 110N for general purpose mortar.

20.5.2.3 Shear strength (clause 3.6.2)

Shear strength of masonry is dependent on the direction of loading, as well as mortar and the unit type. Similar to flexural strength, shear strength is recommended to be determined from tests within BS EN1996; however, values for initial shear strength (unmodified by compressive loads) are tabulated within the National Annex to BS EN1996 (Table NA.5) which are indicated in **Table 20.7**.

Basic shear strengths can be modified by vertical loads to increase the allowable shear stress, as indicated in the equation below from clause 3.6.2.

$$f_{vk} = f_{vko} + 0.4\sigma_d < 0.065f_b \text{ or } f_{vlt}$$

	Initial shear resistance – f_{vk0}		
	M2	M4 to M6	M12
Clay brickwork	0.10	0.20	0.30
Aggregate concrete blockwork	0.10	0.15	0.20

Further guidance is given in NA to BS EN1996-1-1:2005 – Table NA.5 (© BSI, London, UK)
Adapted from and courtesy of the British Standards Institution (BSI)

Table 20.7 Masonry initial shear resistance. Permission to reproduce extracts from British Standards is granted by BSI

	Failure parallel to joints f_{xk1}			Failure perpendicular to joints f_{xk2}		
	M12	M4 to M6	M2	M12	M4 to M6	M2
Clay bricks 7–12% Abs*	0.50	0.40	0.35	1.5	1.1	1.0
100 mm thick aggregate concrete blockwork min 7N**	0.25		0.20	0.60		0.50
140 mm thick aggregate concrete blockwork min 7N**	0.22		0.17	0.53		0.44
215 mm thick aggregate concrete blockwork min 7N**	0.17		0.12	0.41		0.34

Further guidance is given in NA to BS EN1996-1-1:2005 – Table NA.6 (© BSI, London, UK)
Adapted from and courtesy of the British Standards Institution (BSI)
* Further values are given in the NA for clay brickwork outside of the absorption range 7–12%
** Values can also be used for Aerated Aggregate Concrete (AAC) blocks and manufactured stonework of groups 1 and 2

Table 20.8 Masonry flexural strength values. Permission to reproduce extracts from British Standards is granted by BSI

20.5.2.4 Flexural strength (clause 3.6.3)

The flexural strength of masonry is dependent on the type of mortar and the water absorption of clay units, and compressive strength of blockwork. BS EN1996 provides characteristic flexural strengths for both in plane (fkx_1 – failure parallel to bed-joints) and out of plane bending (fkx_2 – failure perpendicular to bed-joints). BS EN1996 suggests that the characteristic flexural strength should be based on test results; however, it provides basic guidance for flexural strengths in mortars being M5 or greater. The National Annex expands on this guidance to relate the flexural strengths to the UK mortar types. **Table 20.8** summarises flexural strength values for typical construction units.

The following design examples are intended to provide some basic guidance on final design to BS EN1996 – part 1. These are typically for unreinforced sections, and relate to clay and blockwork construction. Clause references are provided in italics to the right of the page to allow further reading.

20.5.2.5 Basics

The following examples give some guidance on the 'basics' of masonry design such as the calculation of compressive strengths, factors of safety, etc.

Standard format facing bricks have the following declared properties from the manufacturer:

Gross density	1850 kg/m^3 (High density)
Water absorption	7–12%
Durability etc	F2 and S2
Compressive Strength (fb)	35 N/mm^2 or more
Masonry unit	Category I
Voids	< 25%

For the construction the following classes and mortar strengths are to be used:

Construction class	2
Mortar strength (fm)	M6

Based on the above information, the following values are derived from BS EN1996-1 and the UK National Annex:

Masonry Group	Group 1 (<25% voids)	*Table 3.1*
Equation Constants	$k = 0.50$, $\alpha = 0.70$, $\beta = 0.30$	*Table 3.3 and NA.4*
Material factors of safety	$\gamma_m = 2.70$ for direct of flexural compression	*Table NA.1*
	$\gamma_m = 2.70$ for flexural tension	

Characteristic compressive strength: *Clause 3.6.1.2*

$$f_k = k \times f_b^{\,a} \times f_m^{\,\beta} = 0.50 \times 35^{0.70} \times 6^{0.30}$$

$$= 10.3\ \text{N/mm}^2$$

$$f_d = \frac{10.3}{2.7} = 3.8\ \text{N/mm}^2$$

Characteristic flexural strength: *Table NA.6*

Clause 3.6.3

$$f_{xk1} = 0.40\ \text{N/mm}^2 \quad f_{xd1} = \frac{0.40}{2.70} = 0.15\ \text{N/mm}^2$$

$$f_{xk2} = 1.10\ \text{N/mm}^2 \quad f_{xd2} = \frac{1.10}{2.70} = 0.41\ \text{N/mm}^2$$

100 mm wide dense concrete blocks have the following declared properties from the manufacturer:

Gross density	1450 kg/m³ (High density)
Durability etc	F0
Compressive Strength (fb)	7 N/mm²
Masonry unit	Category I
Voids	<25% (solid block)

For the construction the following classes and mortar strengths are to be used:

Construction class	1
Mortar strength (fm)	M4

Blocks are to be constructed using collar jointed wall construction to clause 8.5.2.3. Therefore, K = 0.8K.

Based on the above information, the following values are derived from BS EN1996-1 and the UK National Annex:

Masonry Group	Group 1 (<25% voids)	*Table 3.1*
Equation Constants	$k = 0.55$, $\alpha = 0.70$, $\beta = 0.30$	*Table 3.3 and NA.4*
Material factors of safety	$\gamma_m = 2.30$ for direct of flexural compression $\gamma_m = 2.30$ for flexural tension	*Table NA.1*

Characteristic compressive strength: *Clause 3.6.1.2*

$$f_k = k \times f_b{}^a \times f_m{}^\beta = 0.55 \times 0.80 \times 7^{0.70} \times 4^{0.30}$$

$$= 2.60\ \text{N/mm}^2$$

$$f_d = \frac{2.60}{2.30} = 1.13\ \text{N/mm}^2$$

Characteristic flexural strength: *Table NA.6*

$$f_{xk1} = 0.25\ \text{N/mm}^2 \quad f_{xd1} = \frac{0.25}{2.30} = 0.11\ \text{N/mm}^2$$ *Clause 3.6.3*

$$f_{xk2} = 0.60\ \text{N/mm}^2 \quad f_{xd2} = \frac{0.60}{2.30} = 0.26\ \text{N/mm}^2$$

Possible uses:

MX 1	Inner leaves of cavity walls, partitions, etc.
MX 2	Masonry in contact with moisture (e.g. plant rooms, etc.) but not in aggressive soils.

20.5.2.6 Local loading (BS EN1996 clause 6.1.3)

Local loading checks are required where local high loads are expected. This may be where steel or concrete beams are supported on beams or where precast planks land onto inner leaves of masonry construction.

A steel beam carrying 6 kN/m dead load and 5 kN/m imposed load spans 5 m between masonry supports. The beam is supported on a 215 mm thick wall constructed in 10 N blockwork in M6 mortar (see **Figure 20.9**) determine the bearing area required.

Steel beam reaction	(N_{EDC})
Loading factors of safety	1.35 Gk and 1.5 Qk *BS EN1990*

$N_{EDC} = (1.35\ Gk + 1.50\ Qk) \times \text{span}/2 = 39$ kN

From the diagram, the load is located at an eccentricity of 50 mm therefore is 0.23t which is less than t/4.

Masonry values:

Compressive Strength (fb)	10 N/mm² or more
Masonry unit	Category I
Masonry Group	Group 1 (<25% voids)
Construction class	1
Mortar strength (fm)	M6

Therefore characteristic compressive strength is:

$$f_k = k \times f_b{}^a \times f_m{}^\beta = 0.55 \times 10^{0.70} \times 6^{0.30} = 4.72\ \text{N/mm}^2$$

Material factors of safety γm = 2.30 for direct of flexural compression

$$f_d = \frac{f_k}{\gamma_m} = \frac{4.72}{2.30} = 2.05\ \text{N/mm}^2$$

Clause 3.6.1.2
Table NA.1

The enhancement factor for concentrated loads (β) is given by:

$$\beta = \left(1 + 0.3\frac{a_1}{hc}\right)\left(1.5 - 1.1\frac{A_b}{A_{ef}}\right)$$

Equation 6.11

a_1	is the distance to the edge of the wall = 600 mm
hc	is the height of the load = 2700 mm
A_b	bearing area of load = 20 000 mm²
A_{ef}	this is the effective bearing area of the wall at mid-height based on a 60° load spread and determines the effective length of bearing at mid-height (see **Figure 20.10**).

The effective bearing area = 339 485 mm²

A_b / A_{ef}	ratio of bearing area and effective bearing area = 0.06, limited to 0.45

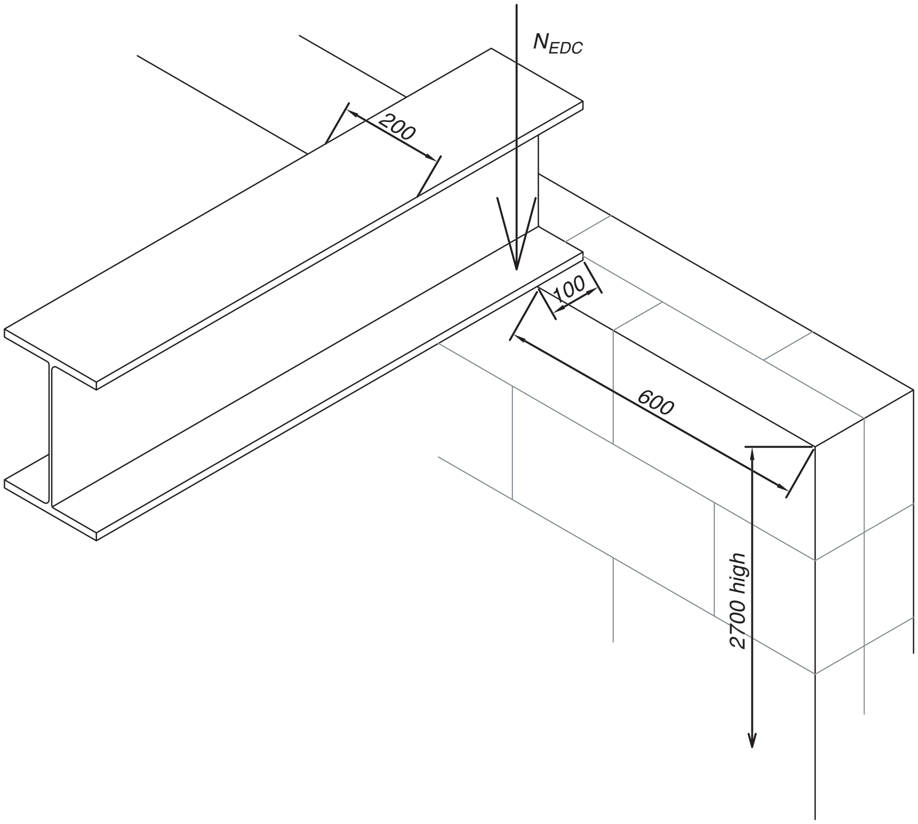


Figure 20.9 Local bearing
The basic requirement for concentrated loads is: $N_{EDC} \leq N_{RDC} = N_{EDC} \leq \beta A_b f_d$

The enhancement factor:

$$\beta = \left(1 + 0.3\frac{600}{2700}\right)\left(1.5 - 1.1\frac{20\,000}{339\,485}\right) = 1.53$$

This is limited to:

$$\beta_{MIN} = 1.0 \; \beta_{MAX} = 1.25 + \frac{a_1}{2hc}$$

$$= 1.36 \; \beta_{MAX} = 1.50 \qquad \text{Equation 6.11}$$

Therefore β maximum is 1.36, hence bearing resistance is given as:

$$N_{RDC} = \beta A_b f_d = \frac{1.36 \times 20\,000 \times 2.05}{1000}$$

$$= 55.8\text{kN} > 39\text{kN} \therefore \text{ok} \qquad \text{Equation 6.10}$$

A 450 mm square masonry pier is constructed in class B engineering bricks in a M6 mortar. The pier carries 300 kN at a nominal eccentricity (<0.25t) (see **Figure 20.11**) determine required bearing.

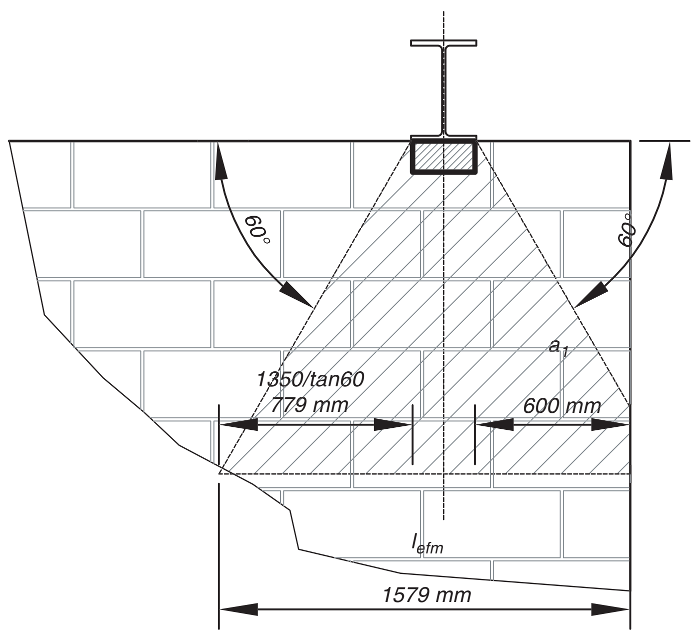


Figure 20.10 Local bearing *Clause 6.1.3 and Figure 6.2*

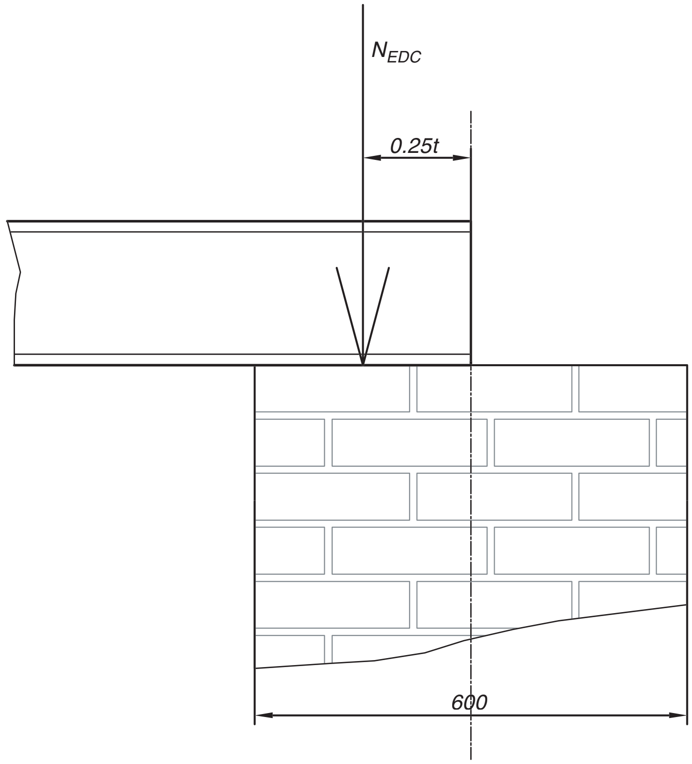


Figure 20.11 Local bearing The basic requirement for concentrated loads is: $N_{EDC} \leq N_{RDC} = N_{EDC} \leq \beta A_b f_d$ *Clause 6.1.3*

Masonry Resistance (N_{RDC})

Compressive Strength (fb)	50 N/mm^2
Masonry unit	Category I
Masonry Group	Group 1 (<25% voids)
Construction class	1
Mortar strength (fm)	M6

Therefore characteristic compressive strength is:

$$f_k = k \times f_b{}^{a} \times f_m{}^{\beta} = 0.50 \times 50^{0.70} \times 6^{0.30} = 13.23 \text{ N/mm}^2$$

Material factors of safety γm = 2.30 for direct of flexural compression

$$f_d = \frac{f_k}{\gamma_m} = \frac{13.23}{2.30} = 5.75 \text{ N/mm}^2 \qquad \text{Clause 3.6.1.2 Table NA.1}$$

Assume initially $\beta = 1.25$, therefore determine A_b

$$N_{EDC} = N_{RDC} = \beta A_b f_d \therefore A_b = \frac{300 \times 1000}{1.25 \times 5.75}$$

$$= 41739 \text{ mm}^2 \qquad \text{Equation 6.10}$$

Assuming 215 mm wide padstone bearing therefore bearing length:

$$L_{bearing} = \frac{41739}{215} = 194 \text{ mm lg}$$

Assume min 215 mm square padstone as bearing, therefore *Ab* = 46 225 mm^2

Check concentrated load (β) factor:

$$\beta = \left(1 + 0.3\frac{a_1}{hc}\right)\left(1.5 - 1.1\frac{A_b}{A_{ef}}\right) \qquad \textit{Equation 6.11}$$

a_1 is the distance to the edge of the wall = (450 – 215)/2 = 117.5 mm

hc is the height of the load = 3000 mm

A_b bearing area of load = 46 225 mm^2

A_{ef} effective bearing area of the wall at mid-height

Bearing will be limited to the area of the column (**Figure 20.12**).

L_{eff} = 450 mm, t = 450 mm, A_{ef} = 202 500 mm^2

A_b / A_{ef} ratio of bearing area and effective bearing area = 0.228

$$\beta = \left(1 + 0.3\frac{117.5}{3000}\right)(1.5 - 1.1 \times 0.228) = 1.26 \quad \beta_{MAX}$$

$$= 1.25 + \frac{a_1}{2hc} = 1.27$$

By inspection assumed β < 1.26 so okay as greater than assumed 1.25 value.

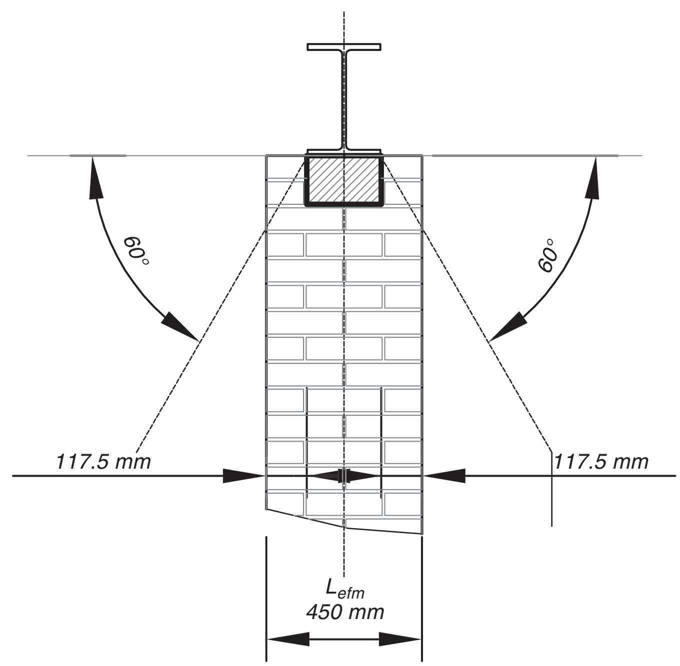


Figure 20.12 Local bearing *Clause 6.1.3 and Figure 6.2*

20.5.2.7 Vertical loading (clause 5.5.1)

Vertical load resistance should be checked for load-bearing walls and columns.

A two-storey house is constructed in load-bearing masonry of cavity construction. The inner leaf is 100 mm wide blockwork (7 N in M4 mortar) and an outer leaf of 100 mm clay brickwork.

The inner leaf is tied to timber floors and carries the following loads:

Above first floor = 11 kN/m Gk and 3.5kN/m Qk at first floor

First floor = 6 kN/m Gk and 8 kN/m Qk at first floor

SW of wall below first floor = 7.5 kN/m

Check the resistance of the inner leaf.

Basic requirement:

$N_{ED} \leq N_{RD} = N_{ED} \leq \varphi t f_d$ *Clause 6.1.2.1 Equation 6.1 & 6.2*

Loading:

$N_{ED\text{-innerleaf}} = (1.35\ Gk + 1.50\ Qk) = 20\ kN/m$ *BS EN 1990*

$N_{ED\text{-innerleaf-firstfloor}} = (1.35\ Gk + 1.50\ Qk) = 21\ kN/m$

$N_{ED\text{-innerleaf-SW}} = (1.35\ Gk) = 10\ kN/m$

Total N_{ED} at base of wall = 51 kN/m

Assumptions/Specification requirements:

Masonry unit	Category I
Masonry Group	Group 1 (<25% voids)
Construction classification	Class 2
Design masonry resistances	

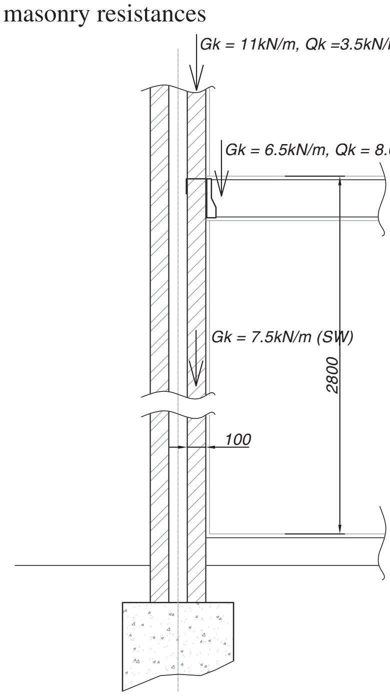


Figure 20.13 Vertical loading

Inner leaf *Clause 3.6.1.2 Table NA.1*

$$f_k = k \times f_b^{\,a} \times f_m^{\,\beta} = 0.55 \times 7^{0.70} \times 4^{0.30}$$

$$= 3.25\ N/mm^2$$

$$f_d = \frac{f_k}{\gamma_m} = \frac{3.25}{2.70} = 1.20\ N/mm^2$$

Outer leaf (assumed as 20N facing bricks in M4 mortar)

$f_k = 6.2\ N/mm^2$

Capacity reduction factor ϕ, due to slenderness and eccentricity

Slenderness:

Effective thickness, $t_{eff} = \sqrt[3]{K_{tef} t_1^3 + t_2^3}$ *Clause 5.5.1.3 Equation 5.11*

K_{tef} is the ratio of the modulus of elasticity for the inner and outer leaves.

E inner leaf = $K_E f_k$, where K_E is taken as 1000 for the UK = 3250 N/mm² *Clause 3.7.2*

E outer leaf = 6200 N/mm² *NA clause 2.9*

$$\frac{E_{outer}}{E_{inner}} = \frac{6200}{3250} = 1.90 < 2.0$$

$$t_{eff} = \sqrt[3]{K_{tef} t_1^{\,3} + t_2^{\,3}} = \sqrt[3]{1.90 \times 100^3 + 100^3} = 142mm$$

Effective height, $h_{ef} = \rho_n h$ *Clause 5.5.1.2 Equation 5.2*

ρ_2 assumed to be 1.0 – timber floors effectively tied to inner leaf, bearing at least 2/3 of the wall thickness

ρ_3 & ρ_4 assumed to be 1.0 – no stiffening walls local to masonry wall

$$\frac{h_{ef}}{t_{ef}} = \frac{2800}{142} = 19.7$$ *Clause 5.5.1.4*

This is less than 27 therefore the wall is within slenderness limits for walls with mainly vertical loading.

The capacity reduction factor ϕ is determined at the top/bottom of the wall and the mid-point, with the minimum value taken for the design.

$$\phi = 1 - 2\left(\frac{e_1}{t}\right)$$ *Clause 6.1.2.2 Equation 6.4*

Eccentricity at the top of bottom of the wall, e_i

$$e_i = \frac{M_{id}}{N_{id}} + e_{he} + e_{init} \geq 0.05t$$ *Clause 6.1.2.2 Equation 6.5*

M_{id} moment in wall due to eccentricity of the floor connection to the wall.

Taken as the first floor load acting at an eccentricity of t/6.

$$M_{id} = \frac{20 \times 100}{6} = 333 \text{ kNmm/m}$$

N_{id} Design value at top (or bottom) of wall = 51.0 kN

e_{he} eccentricity due to horizontal loads = 0 kN

e_{init} eccentricity due to initial imperfections, taken as h/450 = 6.2mm *Clause 5.5.1.1*

$$e_i = \frac{M_{id}}{N_{id}} + e_{he} + e_{init} = \frac{333}{51} + 0 + 6.2 \quad \text{where... } 0.05t = 5\text{mm}$$

$$= 12.9 \text{ mm} \geq 0.05t$$

Reduction factor at top/bottom of wall =

$$\phi_i = 1 - 2\frac{e_i}{t} = 1 - 2\frac{12.9}{100} = 0.74$$

Small plan area factor check is applicable where the cross sectional area of the wall is less than 0.1 m^2, with the modification factor given below:

Area = 0.1 × 1.0 = 0.1 m^2 ≥ 0.1 m^2
∴ (0.7 + 0.3A) = 1.0 *Equation 6.3*

Eccentricity at the middle of the wall, e_m

$$e_{mk} = e_m + e_k = \frac{M_{md}}{N_{mid}} + e_{hm} + e_{init} + e_k \geq 0.05t$$ *Equation 6.6*

M_{id} assumed to be a point of contraflexure, therefore = 0 kNm

N_{id} Design value at top (or bottom) of wall = 50 kN

e_m eccentricity due to vertical loads

$$e_m = \frac{M_{md}}{N_{mid}} = 0$$

e_{he} eccentricity due to horizontal loads = 0 kN

e_{init} eccentricity due to initial imperfections, taken as h/450 = 6.2 mm

e_k eccentricity due to creep, given as

$$e_k = 0.002\phi_\infty \frac{h_{ef}}{t_{ef}}\sqrt{te_m}$$ *Equation 6.8* *Clause 6.1.2.2*

ϕ_w = 1.5, final creep coefficient for aggregate concrete blocks *Table NA.7*

However as slenderness is less than 27, creep may be ignored. *NA to BS EN1996:1*

As $e_m = 0$, $e_k = 0$

$$e_{mk} = \frac{M_{md}}{N_{mid}} + e_{hm} + e_{init} + e_k = 0 + 0 + 6.2 + 0$$

$$= 6.2\text{mm} \geq 0.05t \quad \frac{e_{mk}}{t} = 0.062$$

Annex G provides values of ϕ_m, based on E values as a function of f_k. *Annex G*

For E = 1000f_k figure G.1 is used *Figure G.1*

For this example ϕ_m is approximately equal to 0.61

Limiting value of ϕ_{min} is given as = 0.61 (based on middle).

Compressive resistance:

$$N_{RD} = \phi t f_d = \frac{0.61 \times 100 \times 1000 \times 1.20}{1000} = 73 \text{ kN/m} > N_{ED} \therefore \text{ok}$$

A three-storey building is constructed in load-bearing masonry construction. An internal partition is constructed using 21 5mm wide blockwork (15 N in M4 mortar).

The height of the lowest storey is 4 m high. The floors of the construction are precast concrete and sit on either side of the internal wall (see **Figure 20.14** and **20.15**).

The wall carries the following loads:

Above first floor = 46 kN/m Gk and 20 kN/m Qk

First floor = 16 kN/m Gk and 14 kN/m Qk (8 kN/m Gk and 7 kN/m Qk on each side)

SW of wall below first floor = 14 kN/m

Check the resistance of the inner and outer leaves.

Basic requirement:

$N_{ED} \leq N_{RD} = N_{ED} \leq \phi t f_d$ *Clause 6.1.2.1* *Equation 6.1 & 6.2*

Loading:

$N_{ED\text{-innerleaf}}$ = (1.35 Gk + 1.50 Qk) = 92 kN/m
$N_{ED\text{-innerleaf-firstfloor}}$ = (1.35 Gk + 1.50 Qk) = 21 kN/m
$N_{ED\text{-innerleaf-firstfloor}}$ = (1.00 Gk) = 8 kN/m
>$N_{ED\text{-innerleaf-SW}}$ = (1.35 Gk) = 19 kN/m *BS EN1990*

Total N_{ED} at base of wall = 152 kN/m

Assumptions / Specification requirements:

Masonry unit	Category I
Masonry Group	Group 1 (<25% voids)
Construction classification	Class 2

Design masonry resistances

Internal partition walls (collar jointed construction) *Clause 3.6.1.2*

$$f_k = k \times f_b^{a} \times f_m^{\beta} = 0.55 \times 0.8 \times 15^{0.70} \times 4^{0.30}$$ *Table NA.1*

$$= 4.4\ \text{N/mm}^2$$

$$f_d = \frac{f_k}{\gamma_m} = \frac{4.40}{2.70} = 1.63\ \text{N/mm}^2$$

Capacity reduction factor ϕ, due to slenderness and eccentricity

Slenderness:

Assume wall is constructed using as collar jointed wall with sufficient ties and cement fill between cavity to assume an effective thickness equal to the wall thickness (not two separate leaves).

Effective thickness, $t_{eff} = 215$ mm *Clause 5.5.1.3*

Effective height, $h_{ef} = \rho_n h$ *Clause 5.5.1.2* *Equation 5.2*

ρ_2 assumed to be 0.75 based on precast concrete floor planks on both sides.

The wall is 3 m long and stiffened on one vertical edge as well as the top and the bottom, therefore ρ_3 the stiffening effects of the wall can be used. Therefore for h < 3.5l, ρ_3 =

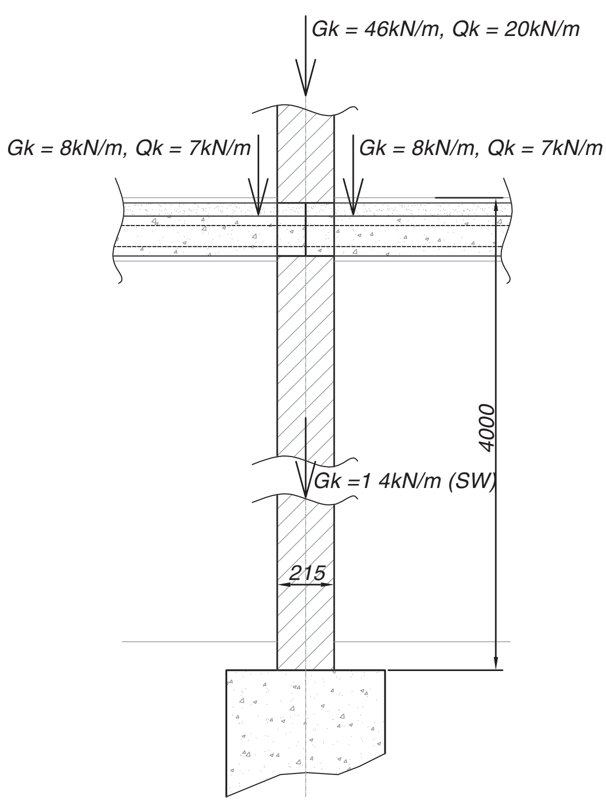


Figure 20.14 Vertical loading

$$\rho_3 \frac{1}{1+\left[\frac{\rho_2 h}{3l}\right]^2}\rho_2$$ *Clause 5.5.1.2* *Equation 5.6*

therefore, $$\rho_3 \frac{1}{1+\left[\frac{0.75\times 4000}{3\times 3000}\right]^2}\times 0.75 = 0.68$$

$\therefore h_{ef} = \rho_n h = 0.75 \times 0.68 \times 4000 = 2040$ mm

$$\frac{h_{ef}}{t_{ef}} = \frac{2040}{215} = 9.5$$

This is less than 27 therefore the wall is within slenderness limits.

The capacity reduction factor ϕ is determined at the top/bottom of the wall and the mid-point, with the minimum value taken for the design.

$$\phi = 1 - 2\left(\frac{e_1}{t}\right)$$ *Clause 6.1.2.2* *Equation 6.4*

Eccentricity at the top of bottom of the wall, e_i

$$e_i = \frac{M_{id}}{N_{id}} + e_{he} + e_{init} \geq 0.05t$$ *Clause 6.1.2.2* *Equation 6.5*

M_{id} moment in wall due to in balance of floor loads, assuming planks have an eccentricity of 50 mm =

$$M_d = (21-8)\times 50 = 650\ \text{kN/m}$$

N_{id} Design value at top (or bottom) of wall = 152 kN

e_{he} eccentricity due to horizontal loads = 0 kN

e_{init} eccentricity due to initial imperfections, taken as h/450 = 8.9 mm *Clause 5.5.1.1*

$$e_i = \frac{M_{id}}{N_{id}} + e_{he} + e_{init}$$ where... 0.05t = 5 mm

$$= \frac{650}{152} + 0 + 8.9 = 13.2\ \text{mm} \geq 0.05t$$

Reduction factor at top/bottom of wall =

$$\phi_i = 1 - 2\frac{e_i}{t} = 1 - 2\frac{13.2\ \text{mm}}{215} = 0.88$$

Small plan area factor check:

Area = 0.215 × 1.0 = 0.215 m² ≥ 0.1 m² *Equation 6.3*

∴ (0.7 + 0.3A) = 1.0

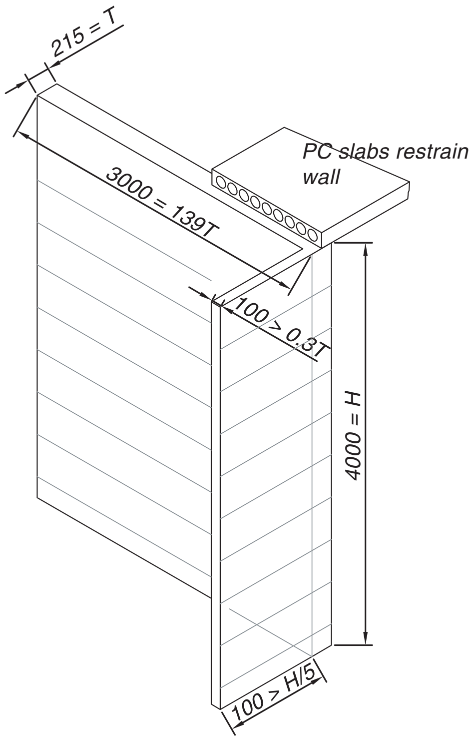


Figure 20.15 Vertical loading

Eccentricity at the middle of the wall, e_m

$$e_{mk} = e_m + e_k = \frac{M_{md}}{N_{mid}} + e_{hm} + e_{init} + e_k \geq 0{,}05t$$

Equation 6.6

M_{id} assumed to be a point of contraflexure, therefore = 0 kNm

N_{id} Design value at top (or bottom) of wall = 150 kN

e_m eccentricity due to vertical loads

$$e_m = \frac{M_{md}}{N_{mid}} = 0$$

e_{he} eccentricity due to horizontal loads = 0 kN

e_{init} eccentricity due to initial imperfections, taken as h/450 = 8.9 mm

e_k eccentricity due to creep, given as

$$e_k = 0.002\phi_\infty \frac{h_{ef}}{t_{ef}} \sqrt{te_m}$$

Equation 6.8
Clause 6.1.2.2

$\phi_w = 1.5$, final creep coefficient for aggregate concrete blocks.

Table NA.7

However, as slenderness is less than 27, creep may be ignored.

NA to BS EN1996:1

As $e_m = 0$, $e_k = 0$

$$e_{mk} = \frac{M_{md}}{N_{mid}} + e_{hm} + e_{init} + e_k$$

$$= 0 + 0 + 6.2 + 0 = 8.9 \text{ mm} < 0.05t \text{ (11 mm)therefore use}$$

$$0.05t \quad \frac{e_{mk}}{t} = 0.05$$

Annex G provides values of ϕ_m, based on E values as a function of f_k.

Annex G
Figure G.1

For E = 1000f_k figure G.1 is used

For this example $\phi_m = 0.66$

Limiting value of ϕ_{min} is given as = 0.66 (based on middle).

Compressive resistance:

$$N_{RD} = \phi t f_d = \frac{0.66 \times 215 \times 1000 \times 1.60}{1000}$$

$$= 234 \text{ kN/m} > N_{ED} \therefore \text{ok}$$

20.5.2.8 Lateral loading (clause 5.5.5)

Lateral loading on masonry panels depends on the wall thickness (elastic modulus) and the geometry and support conditions of the panel.

A building is constructed in load-bearing masonry of cavity construction. The inner leaf is 100 mm wide dense blockwork (7N in M4 mortar) and an outer leaf of 100 mm wide clay brickwork (7–12% abs – 20 N in M4 mortar) (see **Figure 20.15**).

Above first floor = 10 kN/m Gk and 5 kN/m to inner leaf, 6 kN/m Gk to outer leaf

Wind load on panel = 1.00 kN/m²

Panel Dimensions

Height of wall (h) = 2800 mm

Length of wall (L) = 3000 mm

Simple supports to all sides

Annex E

Annex E Wall condition E

Figure E.1

Check the resistance of the inner and outer leaves.

Vertical Loading:

BS EN1990

$N_{ED\text{-}innerleaf}$ = (1.00 Gk) = 10 kN/m

$N_{ED\text{-}outerleaf}$ = (1.00 Gk) = 6 kN/m

Considering γ_f for favourable loading conditions.

Assumptions / Specification requirements:

Masonry unit	Category I
Masonry Group	Group 1 (<25% voids)
Construction classification	Class 2

Flexural strengths of masonry

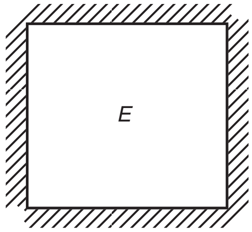


Figure 20.16 Lateral loading

Inner leaf of blockwork (100 mm thick) etc.

Bending parallel $fk_{x1} = 0.25$ *Table NA.6*

Bending perpendicular $fk_{x2} = 0.60$ *Table NA.1*

Factor of safety $\gamma m = 2.70$

Design resistances for inner leaf *Clause 5.5.5 – note (7)*

$$f_k = k \times f_b^{\,a} \times f_m^{\,\beta} = 0.55 \times 7^{0.70} \times 4^{0.30}$$ *Table Na.1*

$$= 3.25 \text{ N/mm}^2$$

$$f_d = \frac{f_k}{\gamma_m} = \frac{3.25}{2.70} = 1.20 \text{ N/mm}^2$$

$$f_{dx1} = \frac{f_{kx1}}{\gamma_m} = \frac{0.25}{2.70} = 0.09 \text{ N/mm}^2$$

$$f_{dx2} = \frac{f_{kx1}}{\gamma_m} = \frac{0.60}{2.70} = 0.22 \text{ N/mm}^2$$

Orthogonal Ratio $\mu = 0.41$

Outer leaf of brickwork (100 mm thick), etc.

Bending parallel $fk_{x1} = 0.40$ *Table NA.6*

Bending perpendicular $fk_{x2} = 1.10$ *Table NA.1*

Factor of safety $\gamma m = 2.70$

Design resistances for outer leaf *Clause 5.5.5 – note (7)*

$$f_k = k \times f_b^{\,a} \times f_m^{\,\beta} = 0.50 \times 20^{0.70} \times 4^{0.30}$$ *Table NA.1*

$$= 6.17 \text{ N/mm}^2$$

$$f_d = \frac{f_k}{\gamma_m} = \frac{6.17}{2.70} = 2.28 \text{ N/mm}^2$$

$$f_{dx1} = \frac{f_{kx1}}{\gamma_m} = \frac{0.40}{2.70} = 0.15 \text{ N/mm}^2$$

$$f_{dx2} = \frac{f_{kx1}}{\gamma_m} = \frac{1.10}{2.70} = 0.41 \text{ N/mm}^2$$

Orthogonal Ratio $\mu = 0.36$

Basic requirement for lateral loads on panels:

$$M_{EDl} \le M_{RDl}$$

$$\therefore \mu\alpha W \gamma_f L^2 \le \left\lfloor f_{dxlapp} \right\rfloor Z = \left[f_{dx1} + \sigma_d \right] Z$$

Clause 5.5.5 4- note (7) and equation 6.1

$$M_{ED2} \le M_{RD2} \therefore \mu\alpha W \gamma_f L^2 \le f_{dx2} Z$$

Allowable panel size:

Limiting panel sizes are given graphically in BS EN1996 part 1, Annex F, figures F.1 to F.3

This annex provides information on limiting panel size based on H/t and L/t ratios. Within this example t is based on the effective thickness of the masonry.

Effective thickness, $t_{eff} = \sqrt[3]{K_{tef} t_1^{\,3} + t_2^{\,3}}$ *Clause 5.5.1.3 Equation 5.11*

K_{tef} is the ratio of the modulus of elasticity for the inner and outer leaves.

E inner leaf = $K_E f_k$, where K_E is taken as 1000 for the UK = 3250 N/mm^2 *Clause 3.7.2*

E outer leaf = 6200 N/mm^2 *NA clause 2.9*

$$\frac{E_{outer}}{E_{inner}} = \frac{6200}{3250} = 1.90 < 2.0$$

$$t_{eff} = \sqrt[3]{K_{tef} t_1^{\,3} + t_2^{\,3}} = \sqrt[3]{1.90 \times 100^3 + 100^3} = 142 \text{ mm}$$

$$\text{Height to thickness ratio} = \frac{H}{t_{eff}} = \frac{2800}{142} = 19$$

$$\text{Length to (effective) thickness ratio } \frac{L}{t_{eff}} = \frac{3000}{142} = 21$$

Panel within limits of graph F.1 – panel ok *Annex F Figure F.1*

Inner leaf resistance – 100 mm blockwork

Elastic Modulus: $Z = \frac{t^2 L}{6} = \frac{100^2 \times 1000}{6} = 1.66 \times 10^6 \text{mm}^3$

Vertical load per unit area: $\sigma_d = \frac{10 \times 1000}{100 \times 1000} = 0.10 \text{ N/mm}^2$ *Note to equation 6.1.6*

NB: the vertical load at mid height including the block work SW may be considered depending on the panel type. For this example the SW of the wall leafs has been ignored.

$$\sigma_{dLIM} = 0.20 \times f_d = 0.24 \text{ N/mm}^2$$

$$\therefore \text{OK}$$

Parallel resistance: $M_{RD2} = f_{dx2}Z = 0.22 \times \frac{1.66 \times 10^6}{10^6}$ *Equation 6.16*

$= 0.36$ kNm

Perpendicular resistance: $M_{RD2} = f_{dx2}Z = 0.22 \times \frac{1.66 \times 10^6}{10^6}$ *Equation 6.15*

$= 0.36$ kNm

Modified orthogonal ratio: $\mu_{modified} = \frac{f_{xk1} + \gamma_f \sigma_d}{f_{xk2}}$ *Clause 5.5.5 4- note (7)*

$= \frac{0.09 + 0.10}{0.22} = 0.84$

Outer leaf resistance – 100 mm brickwork

Elastic Modulus: $Z = \frac{t^2 L}{6} = \frac{100^2 \times 1000}{6}$

$= 1.66 \times 10^6 \text{mm}^3$

Vertical load per unit area: $\sigma_d = \frac{6 \times 1000}{100 \times 1000} = 0.06 \text{ N/mm}^2$ *Note to equation 6.1.6*

$\sigma_{dLIM} = 0.20 \times f_d = 0.46 \text{ N/mm}^2$

$\therefore$ OK

Parallel resistance: $M_{RD1} = [f_{dx1} + \sigma_d]Z$ *Equation 6.16*

$= [0.15 + 0.06] \times \frac{1.66 \times 10^6}{10^6}$

$= 0.35$ kNm

Perpendicular resistance: $M_{RD2} = f_{dx2}Z$ *Equation 6.15*

$= 0.41 \times \frac{1.66 \times 10^6}{10^6} = 0.68$ kNm

Modified orthogonal ratio: $\mu_{modified} = \frac{f_{xk1} + \gamma_f \sigma_d}{f_{xk2}}$ *Clause 5.5.5 4- note (7)*

$= \frac{0.15 + 0.06}{0.41} = 0.51$

Lateral load taken by outer leaf, based on proportion of load resistance and therefore stiffness:

$W_{INNERLEAF} \therefore = \frac{0.36}{0.68 + 0.36} = 35\%$

$\therefore = 1.00 \times \frac{35}{100} = 0.35 \text{ kN/m}^2$

$W_{OUTERLEAF} \therefore = \frac{0.68}{0.68 + 0.36} = 65\%$

$\therefore = 1.00 \times \frac{65}{100} = 0.65 \text{ kN/m}^2$

Applied bending forces due to wind loads determined from wall support condition E, from BS EN1996 – Annex E, Figure E.1.

Inner leaf

For h/L = 0.93 and Initial orthogonal ratio, $\mu = 0.84$

Bending coefficient, $\alpha = 0.041$ (based on approximate interpolation) *Annex E, Figure E.1*

$M_{ED2\text{-}INNERLEAF} = \alpha W \gamma_f L^2$ *Equation 5.17*

$= 0.041 \times 0.35 \times 1.5 \times 3.00^2$

$= 0.19 \text{ kNm} < 0.36 \text{ kNm}$

$M_{ED2\text{-}INNERLEAF} = \mu\alpha W \gamma_f L^2 = 0.84 \times 0.19$ *Equation 5.18*

$= 0.16 \text{ kNm} < 0.32 \text{ kNm}$

Outer leaf

For h/L = 0.93 and, Initial orthogonal ratio, $\mu = 0.51$

Modified orthogonal ration based on vertical loads:

Bending coefficient, $\alpha = 0.053$ (based on approximate interpolation) *Annex E, Figure E.1*

$M_{ED2\text{-}INNERLEAF} = \alpha W \gamma_f L^2$ *Equation 5.17*

$= 0.053 \times 0.65 \times 1.5 \times 3.00^2$

$= 0.47 \text{ kNm} < 0.68 \text{ kNm}$

$M_{ED2\text{-}INNERLEAF} = \mu\alpha W \gamma_f L^2$ *Equation 5.18*

$= 0.51 \times 0.47$

$= 0.23 \text{ kNm} < 0.35 \text{ kNm}$

By inspection panel is OK.

An internal partition is constructed in 215 mm thick non-load-bearing blockwork. The partition is 4.5 m high and is subject to an internal wind pressure of 0.40 kN/m^2. Check capacity of 10.4N block in M6 mortar.

Assumptions/Specification requirements:

Masonry unit	Category I
Masonry Group	Group 1 (<25% voids)
Construction classification	Class 1

Bending parallel $fk_{x1} = 0.25$ *Table NA.6*

Bending perpendicular $fk_{x2} = 0.75$ *Table NA.1*

Factor of safety $\gamma_m = 2.30$

Design resistances *Clause 5.5.5 – note (7)*

$f_k = k \times f_b^{\ a} \times f_m^{\ \beta}$

$= 0.55 \times 10.4^{0.70} \times 6^{0.30} = 4.85 \text{ N/mm}^2$ *Table Na.1*

$f_d = \frac{f_k}{\gamma_m} = \frac{4.85}{2.30} = 2.10 \text{ N/mm}^2$

$f_{dx1} = \frac{f_{kx1}}{\gamma_m} = \frac{0.25}{2.30} = 0.10 \text{ N/mm}^2$

$f_{dx2} = \frac{f_{kx1}}{\gamma_m} = \frac{0.75}{2.30} = 0.32 \text{ N/mm}^2$

Orthogonal Ratio $\mu = 0.33$

Allowable panel size:

Limiting panel size for a wall supported top and bottom in accordance with Annex F should not exceed 30t.

Height to thickness ratio $= \frac{H}{t} = \frac{4500}{215} = 21$ *Annex F*

Applied loads

The weight of the wall will generate some moment at the base, such that the wall can be assumed to be a propped cantilever.

The basic applied load would be similar to a simply supported beam, in that:

$M_{ED} = 0.125 W_{\gamma f} L^2 = 0.125 \times 0.40 \times 1.5 \times 4.50^2$
$= 1.51 \text{kNm}$

The moment of resistance at the base will reduce the moment within the height of the panel, therefore the resistance of the base is considered first:

Vertical load per unit area at base of wall (assuming density of 1800 kg/m²)

Vertical load per unit area: $SW = 0.215 \times 18 \times 4.5 = 17.41$ kN/m *Note to equation 6.1.6*

$$\sigma_d = \frac{17.41 \times 1000}{215 \times 1000} = 0.08 \text{ N/mm}^2$$

$$\sigma_{dLIM} = 0.20 \times f_d = 0.97 \text{ N/mm}^2 \therefore \text{OK}$$

Elastic Modulus:
$$Z = \frac{t^2 L}{6} = \frac{215^2 \times 1000}{6}$$
$$= 7.70 \times 10^6 \text{mm}^3$$

Parallel resistance:
$$M_{RD1-BASE} = [f_{dx1} + \sigma_d]Z$$
$$= [0.10 + 0.08] \times \frac{7.70 \times 10^6}{10^6}$$ *Equation 6.16*
$$= 1.38 \text{ kNm}$$

Calculate moment within span and determine location (see **Figure 20.17**).

Prop force at top of wall:

$$Vtop = \frac{1.5 \times 0.4 \times 4.5}{2} - \frac{1,38}{4,5}$$
$$= 1,04 \text{ kN} \therefore \text{Shear} = 0 \text{ kN @ } 1.73 \text{ m}$$

$$M_{ED\text{-}SPAN} = 1.04 \times 1.73 - 1.5 \times 0.4 \times \frac{1.73^2}{2} = 0.90 \text{ kNm}$$

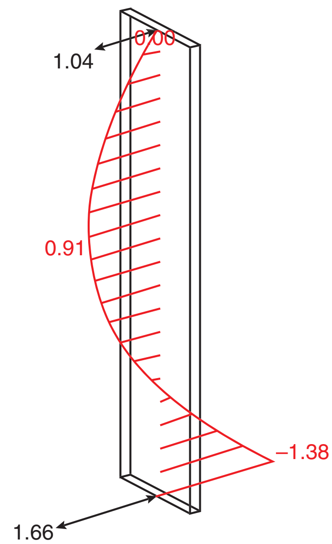


Figure 20.17 Lateral loading

Resistance at 1.84 m from top of wall:

Vertical load per unit area: $SW = 0.215 \times 18 \times 1.73 = 6.70$ kN/m *Note to equation 6.1.6*

$$\sigma_d = \frac{6.70 \times 1000}{215 \times 1000} = 0.03 \text{ N/mm}^2$$

$$\sigma_{dLIM} = 0.20 \times f_d = 0.97 \text{ N/mm}^2 \therefore \text{OK}$$

Parallel resistance:
$$M_{RD1-BASE} = [f_{dx1} + \sigma_d]Z$$ *Equation 6.16*
$$= [0.10 + 0.03] \times \frac{7.70 \times 10^6}{10^6}$$
$$= 1.00 \text{ kNm}$$

Check shear at base of panel.

Design shear force at base of wall, dead load no longer beneficial, therefore $\gamma_f = 1.35$

Basic requirement $V_{ED} \le V_{RD}$ *Clause 6.2*

Applied shear at base of wall:

$$V_{BASE} = \frac{1.5 \times 0.4 \times 4.5}{2} + \frac{1.35 \times 1.38}{4,5} = 1.76 \text{ kN/m}$$

$$V_{ED} = \frac{1.76 \times 1000}{215 \times 1000} = 0.008 \text{ N/mm}^2$$

Design shear resistance:	$f_{vk} = f_{vko} + 0.4\sigma_d < 0.065 f_b or f_{vit}$	*Clause 3.6.2* *Equation 3.5*
	$f_{vk} = f_{vko} + 0.4\sigma_d = 0.15 + (0.4 \times 0.08) = 0.18\ \text{N/mm}^2$	*Table NA.5* *Equation 3.5*
	$f_{vd} = \dfrac{0.18}{2.0} = 0.09\ \text{N/mm}^2$	

Therefore shear OK.

20.5.3 Other aspects of final design

20.5.3.1 Movement

Masonry construction can be intolerant of movement, leading to cracking. Movement can occur for a number of reasons such as:

- *Thermal loads*: Masonry will be subject to changes in thermal loads. This is of particular concern in external facades where direct radiant solar gains may mean temperature changes of 50°C. Movement joints to accommodate thermal loads can be determined from first principles based on a knowledge of the coefficient of thermal expansion of masonry. Typical values are given in **Table 20.9**.
- *Creep and shrinkage*: Creep and shrinkage affect all construction materials which are subject to loading. This is particularly prevalent in masonry where the units and mortars contain moisture. From the National Annex to BS EN1996, table NA.7, the final creep coefficients for all masonry types should be taken as 1.50.
- *Over restraint*: Cracking can occur when masonry is over restrained. This is typically when masonry is closely packed or tied to other construction elements with differing movement criteria. Examples of this are masonry infills to reinforced concrete frames and masonry with concrete slabs cast directly to the upper face.
- *Dimensional or support irregularities*: Cracking in masonry can occur when it is founded or supported by flexible structures or foundations which suffer movement. Poorly designed supports, such as steel beams with incorrect deflection criteria, would allow movement and subsequently cracking. Settlement in foundations may occur for a number of reasons, which can be identified by the type of cracking in the masonry facades. Changes in height are also often changes in stiffness which can lead to cracking.

The difference in coefficient of thermal expansion of clay, calcium silicate and concrete units means that the location of movement joints will vary in different constructions. Guidance is given in the National Annex. Some examples are given in **Table 20.10**. This varies from the previous guidance given in BS 5628 (given in brackets).

Unit type	Coefficient of thermal expansion
Clay brickwork	6 x 10^{-6}/K
Aggregate concrete blockwork	10 x 10^{-6}/K

For further guidance refer to NA to BS EN1996-1, Table NA.7 (© BSI, London, UK)
Adapted from and courtesy of the British Standards Institution (BSI)

Table 20.9 Coefficients of thermal expansion. Permission to reproduce extracts from British Standards is granted by BSI

Movement joints need to be located to ensure that they do not create instability of the structure. A holistic design approach is required when detailing the location of movement joints to ensure that assumptions made in the design are reflected in the construction details. For example, if movement joints are located in the centre of a long wall, the wall cannot be assumed as continuous. The design and specification for movement joints should also consider the following aspects:

- Fire resistance
- Thermal performance
- Acoustic performance.

20.5.3.2 Tolerances

Tolerances in masonry construction are dealt with BS EN1996 – part 2, clause 3.4 and Table 3.1. These are summarised in **Table 20.11**.

20.5.3.3 Health and safety considerations

The design and construction of masonry, as with any material, should consider the health and safety of the operatives on site. The following list gives some guidance, albeit not exhaustive, of considerations which should be made when designing and specifying masonry construction:

Masonry unit type	Distance between joints	Comments
Clay masonry units	15 m (12 m)	This may be increased where bed-joint reinforcement is used. Tables published with the BRC website suggest this may be increased to 18 or even 20 m.
Aggregate concrete blockwork	9 m (6 m)	Applicable when L/H is equal to or less than 3. Similar to clay units, the distance may be increased may where bed-joint reinforcement is used. Tables published with the BRC website suggest this may be increased to 12 or up to 15 m.

Table 20.10 Distance between movement joints

	Maximum deviation (mm)
Verticality of one storey	+/– 20
Verticality of up to three storeys	+/– 50
Straightness in one metre	+/– 10
Straightness in ten metres	+/– 50
Overall thickness of cavity wall	+/– 10

Table 20.11 Tolerances

- Lifting of masonry units
- Working at height
- Temporary stability of walls
- Use of cements and lime mortars.

20.6 References

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20.6.2 Useful web addresses

Brick Development Association – www.brick.org.uk
Building Research Establishment (BRE) – www.bre.co.uk

doi: 10.1680/mosd.41448.0397

Chapter 21

Glass

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The use of glass in buildings poses several challenges to the structural engineer. The aim of this chapter is to provide a unified method for the structural design of glass elements in buildings based on the limit state design philosophy. The chapter identifies the structural performance requirements as functions of the intended application and provides advice on the calculations and prototype testing required for assessing the performance of the candidate design. The chapter also provides advice on material selection and connection design. Further useful sources of information for detailed design and specification are listed.

CONTENTS

21.1 Introduction

The unique combination of transparency, high quality finish, durability and relatively low cost make glass a unique and popular material in buildings. Around 90% of primary glass production is based on the float process developed by the Pilkington brothers in the 1950s. Since then there have been several developments and improvements to the structural and thermal performance of basic float glass. These improvements have led to better performing and larger glass units in facades and to the use of glass as a load-bearing material in roofs, canopies, staircases and floors. The rapid developments in emerging products and the novel uses of glass present a challenge to the structural engineer, particularly on how to design safe and efficient structures with an inherently brittle material and to do so without compromising other constraints such as acoustic performance, thermal performance and architectural intent.

This chapter provides a unified method for the structural design of glass elements based on the limit state design philosophy. Sections 21.2 and 21.3 provide information on structural glass typologies, the performance requirements and the mechanical properties that are important for form and material selection. Section 21.4 lists the loads on glass structures and provides guidelines on the detailed calculations and prototype testing required to satisfy the performance requirements. Finally, Section 21.5 provides simplified rules for glass design including stability problems and connection sizing.

The chapter is not exhaustive, but is intended to provide a basis for initial design, specification and prototype testing. Further sources of information are provided in the text.

21.2 Structural use of glass in buildings

21.2.1 Forms of structural glass

21.2.1.1 Definition

The term 'structural glass' is often misused and is potentially confusing as it implies that glass is either structural or non-structural. For most applications, the use of glass can be described as one of the following:

(a) Primary structure – glass members in this category contribute to the load-bearing capacity of the main structure. Failure of the glass member would compromise the load-bearing capacity of other primary elements and/or other secondary structures and has a very high potential for injury. Glass members in this category include columns, primary floor beams and shear walls.

(b) Secondary structure – where the glass member either contributes to the load-bearing capacity of the secondary structure or where failure of the member has a high potential for injury. Glass members in this category include glass fins, frameless balustrades, top-hung frameless glazing systems, glass floors, glass threads in staircases and overhead glazing.

(c) Infill panels – glass members in this category are normally glass plates that are supported by a sub-frame at two or more edges, such as found in glazed curtain walls, but include non-load-bearing point-supported glazing.

21.2.1.2 Types of glass structures

Over the last 30 years the use of glass has evolved from small flat infill panels in facades to large complex assemblies where the glass elements fulfil a primary structural function. Some examples of novel applications of glass are shown in **Figures 21.1** to **21.4.**

21.2.1.3 Classification by structural behaviour

From a structural point of view it is convenient to consider the flow of forces in each structural glass element. In doing so it is possible to categorise the members as one of the following:

- Struts or ties – where the glass element is predominantly in uniaxial tension or compression (**Figure 21.1(a,b)**).
- Beams – where the members transmit the applied loads in bending about their cross-sectional axes and shear parallel and perpendicular to their cross-section (**Figure 21.2(a,b)**).
- Plates – which transmit the applied loads in biaxial bending and twisting moments and shear forces. This often involves

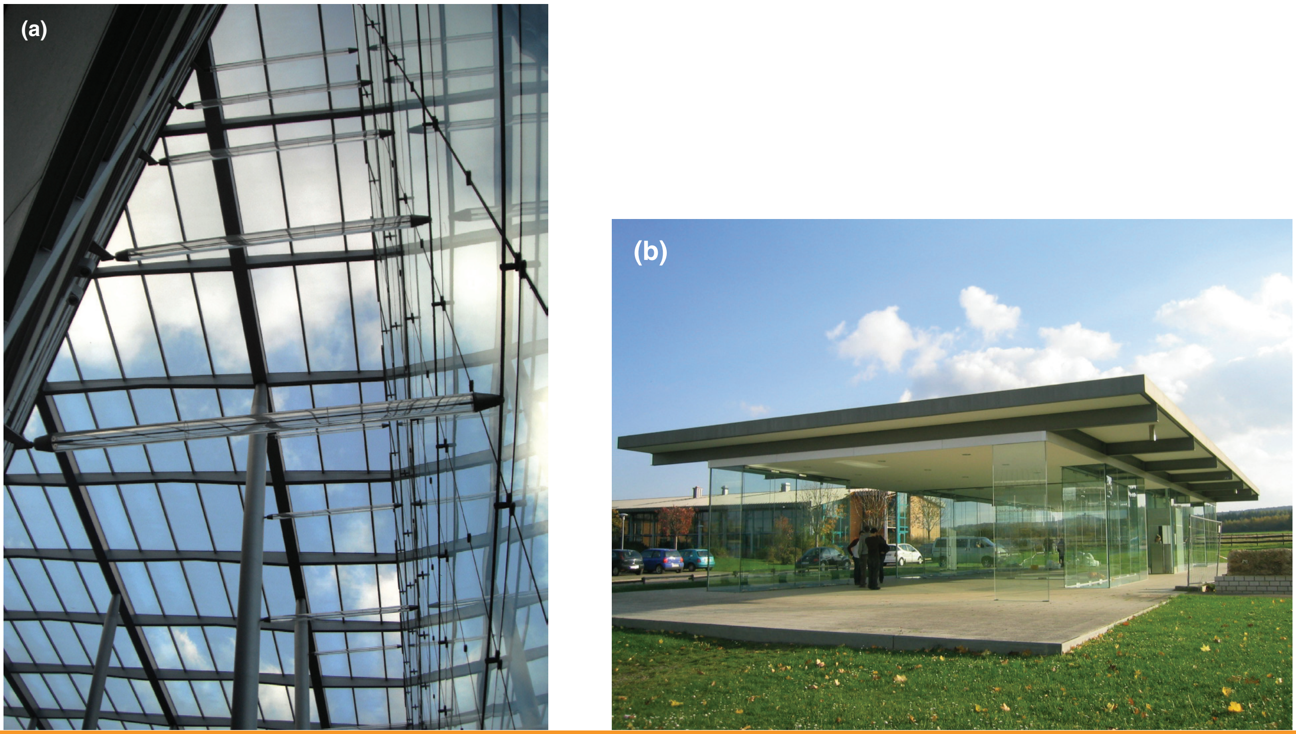

Figure 21.1 Glass struts: (a) glass circular hollow struts providing horizontal restraints to the facade at Tower Place, London; (b) vertical structural glass walls bearing the weight of the steel roof at the Rhienbach Pavilion, Germany

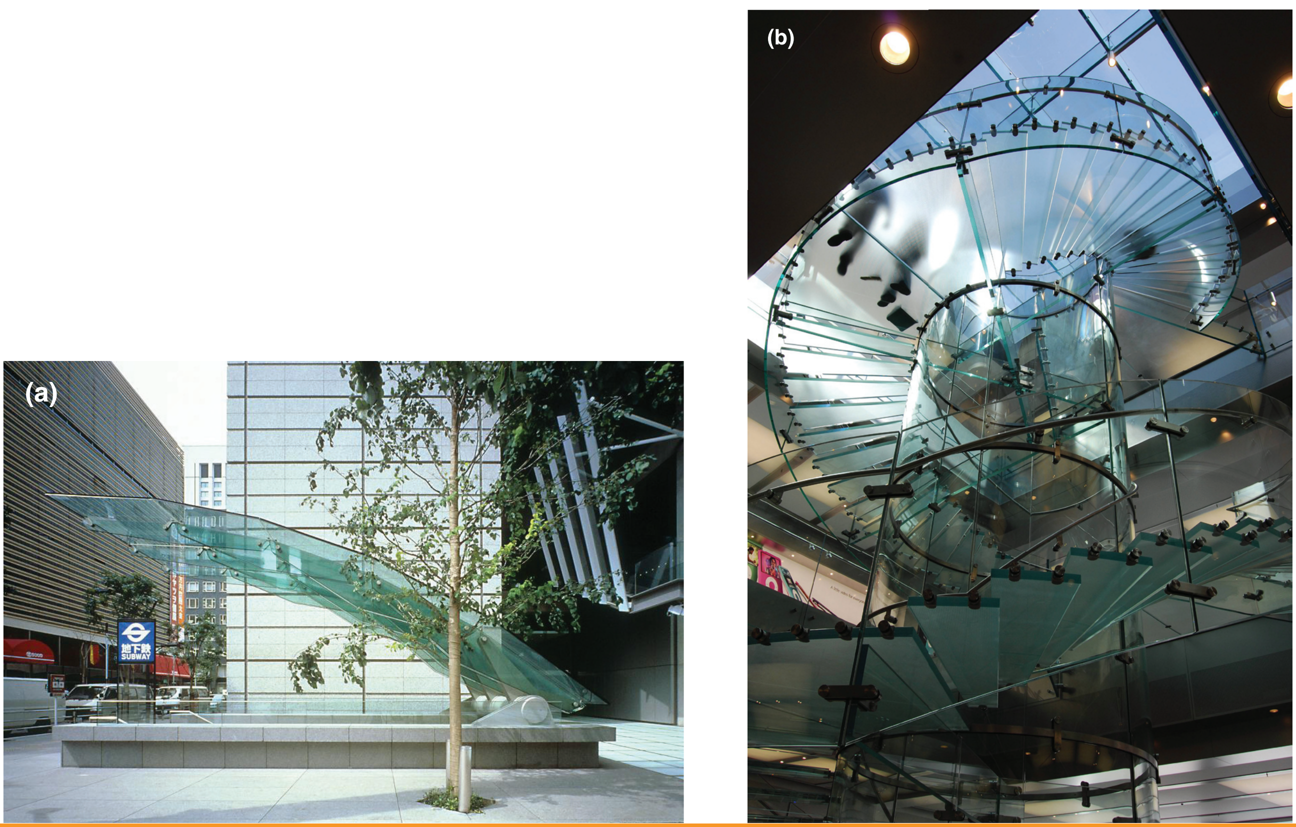

Figure 21.2 Glass beams: (a) cantilevered glass segmented beams in the Yurakucho canopy, Tokyo (image courtesy of Dewhurst Macfarlane Engineers); (b) simply supported glass threads in Apple Store spiral staircase, New York (image courtesy of Eckersley O'Callaghan Engineers)

large deflections (small displacement theory is no longer valid) (**Figure 21.3(a,b)**).

- Shells – which transmit the applied loads as membrane stresses acting on a tangential plane at a given point to the surface (**Figure 21.4**).

In some cases, glass elements may behave in a combined mode, for example, in beam-column action.

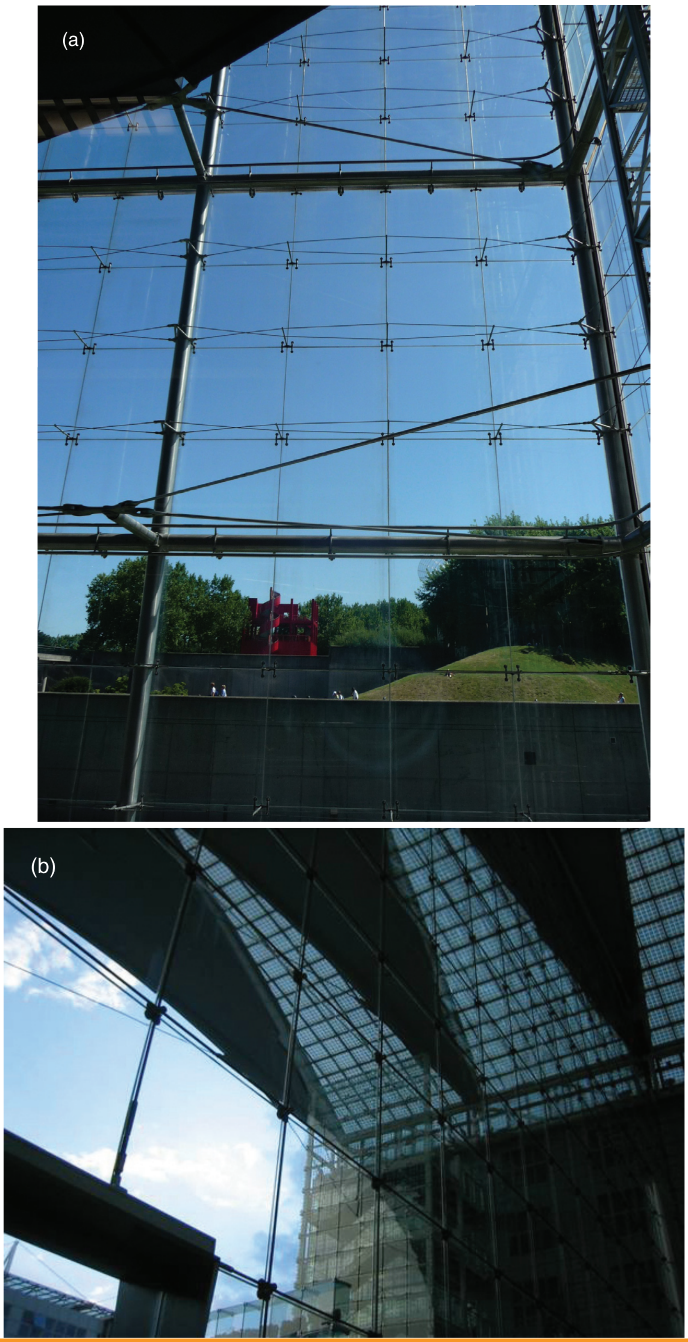

Figure 21.3 Glass plates: (a) frameless glass facade at the at Parc La Villette, Paris; (b) post-tensioned glass facade at Kempinski Hotel, Munich

21.2.1.4 Classification by connection

Over the last 30 years, the trend to increase the architectural transparency of glass assemblies has fuelled rapid developments in glass connections. There are two principal categories of connections for glass: (a) framed glazing connections (also known as linear supports); and (b) frameless glazing connections (sometimes referred to as discrete or point supports). These two can be subdivided further as described below.

Conventional framed glazing

In framed glazing (**Figure 21.5**) a framework of prismatic rectilinear elements known as profiles and generally made of timber, aluminium alloy or steel, is used to support glazing infill panels. The glazing panels are held in position between the profile and an external capping strip. Gaskets made of ethylene propylene diene monomer rubber (EPDM), neoprene

Figure 21.4 Prototype glass shell at the University of Stuttgart, Germany (image courtesy of Werner Sobek Engineers)

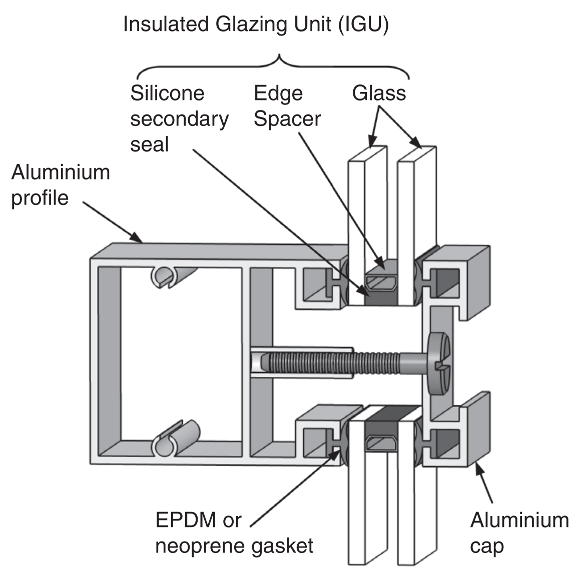

Figure 21.5 Horizontal section through a conventional framed glazing mullion

or silicone, are used to avoid direct contact between the glass and the metallic frame. The gaskets do not provide a rotationally rigid support and the glass units are generally considered to be simply supported. Polyoxymethylene (POM) setting blocks are also used in facades to transfer the self-weight of the vertical glazing units to the horizontal transoms.

In some cases the linear profiles may be used to transmit in-plane forces into the glass (i.e. shell action). This requires a much higher degree of engineering and attention to detail (Haldimann *et al*., 2008).

Structural silicone glazing

In structural silicone glazing (**Figure 21.6**), the capping strip is removed and the glass is bonded to the profiles by means of an elastomeric silicone adhesive. The lateral loads on the glass are transmitted through the silicone adhesive to the profile. The vertical self-weight of the glass is transmitted in bearing through a POM setting block placed on the profile. The gap between adjacent glass units is sealed with a one-component silicone sealant.

Clamped/friction-grip frameless glazing

These connections, also known as patch plate fittings, consist of small rectangular stainless steel or aluminium plates used for clamping glass panels close to the edges of the glass. The clamping force is generated by bolts with oversize holes in the glass, thereby preventing direct bearing of the bolt on the glass. There are two variations of these clamped connections: (1) Low friction clamped fittings (**Figure 21.7(a)**) are used to transfer lateral loads in a similar way to framed glazing, i.e. by introducing a flexible neoprene or EPDM liner between the clamping plate and the glass and using a POM setting block to transfer self-weight. (2) High friction clamped fittings (**Figure 21.7(b)**) are essentially friction-grip connections where the bolts are tightened to a predetermined torque thereby generating a resistance to in-plane forces. The gaskets in the latter are made from stiffer materials (generally aluminium or nylon fibre). Local inserts of these materials are also used in laminated glass to prevent the interlayer from being squeezed out by the clamping force. The substructure to which the clamped fittings are attached can be made of glass (e.g. glass fins).

Bolted frameless glazing

In bolted connections the in-plane and lateral forces to the glass are transmitted as transverse shear and direct forces to the bolt respectively. The bolts are generally made of stainless steel and rely on bushings, made of aluminium, POM or injected two-component mortars, between the bolt and the glass, thereby reduce bearing stress concentrations in the glass.

There are a variety of bolted connections (**Figure 21.8**) ranging from fully rigid bolted connections, where the glass is restrained from rotating out of plane, to fully articulated bolted connections, where the glass is free to rotate with respect to the

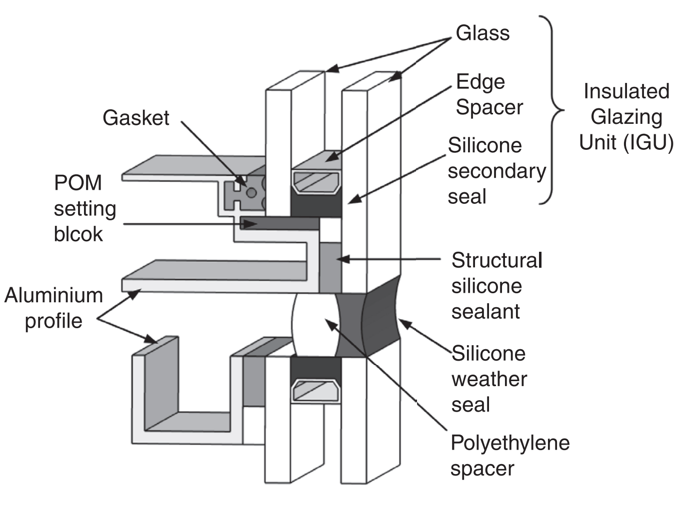

Figure 21.6 Vertical section through a structural silicone glazing transom

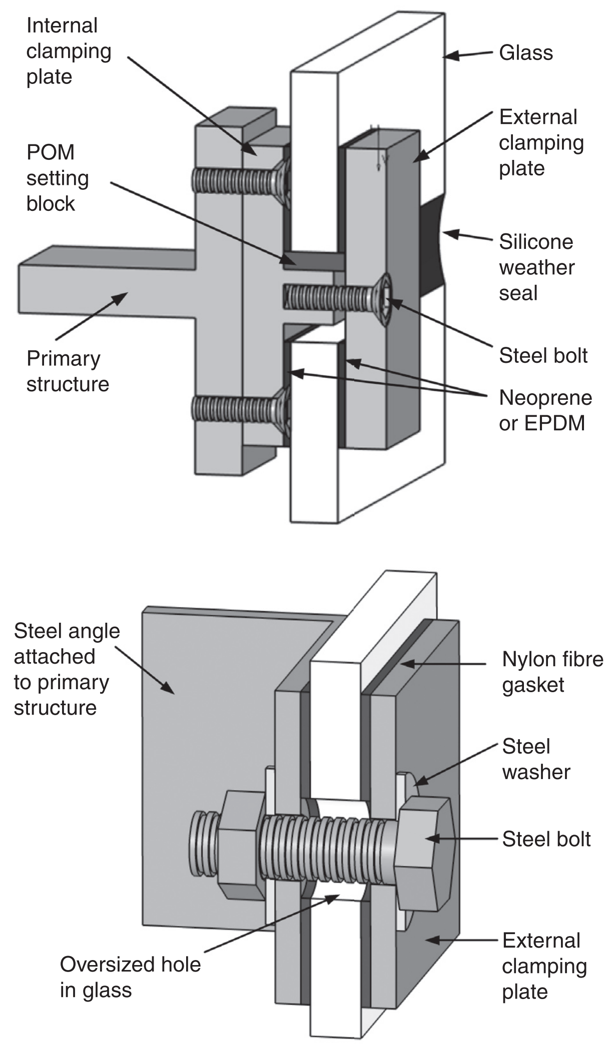

Figure 21.7 Glass plates: (a) low friction clamped connection; (b) high friction (friction grip) clamped connection

bolt. Semi-rigid connections that allow an intermediate degree of resistance to rotation are also available. Bolted glass connections are also distinguishable by the hole profile, for example, through hole, countersunk hole, partial penetration hole.

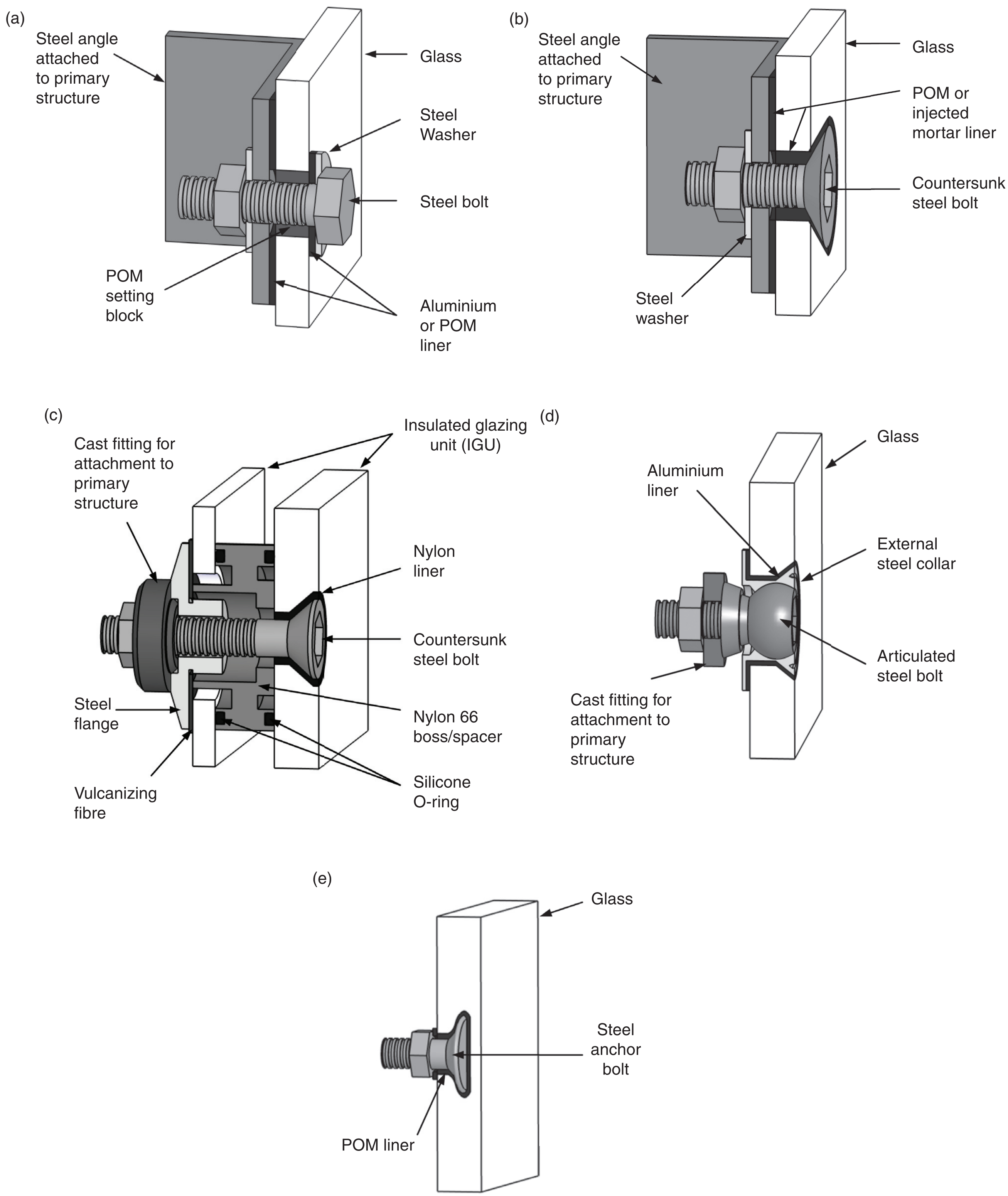

Figure 21.8 Bolted connections: (a) semi-rigid spring plate through-bolt; (b) semi-rigid spring-plate countersunk; (c) planar countersunk; (d) fully articulated countersunk; (e) partial penetration glass anchor

The bolts are often connected to a cast node (spider fitting) that is in turn supported by a sub-frame or cable trusses.

Adhesive frameless

The adhesives in these connections transfer the principal loads from glass-to-glass or glass-to-metal. The adhesives used are generally thermosets (e.g. UV-cured acrylics, two-component acrylics and two-component epoxies) which are stronger and stiffer than the aforementioned elastomeric structural silicone. The principal advantages of adhesive connections over bolted connections is that they produce lower stress concentrations and they require little or no preparation, unlike bolted connection where holes must be drilled into the glass (**Figure 21.9**).

There are, however, uncertainties on the long-term performance of some of these adhesives and their performance in fire tends to be poor. As a result structural adhesive connections are the subject of ongoing research.

Hybrid connections and other novel connections

A new generation of hybrid connections are emerging which involve a combination of bolting and bonding. It is important to note that when bolts in clearance holes and adhesives are used simultaneously, the effective stiffness of the adhesive will tend to be significantly greater than that of the bolts. It is, therefore, sensible to assume that all loads will initially be transferred through the adhesive and the bolts will only come into effect when the adhesive has deformed significantly which is often at failure. Nevertheless, bolts can enhance the load-bearing capacity of an adhesive joint by, for example, using an adhesively bonded friction grip connection as shown in **Figure 21.10(a)**.

It is also possible to use bonding and bolting in series by, for example, bonding a glass to a steel plate which in turn is bolted to the steel substructure (**Figure 21.10(b)**). Other novel methods such as soldered glass joints (**Figure 21.10(c)**) are emerging, and are the subject of ongoing research.

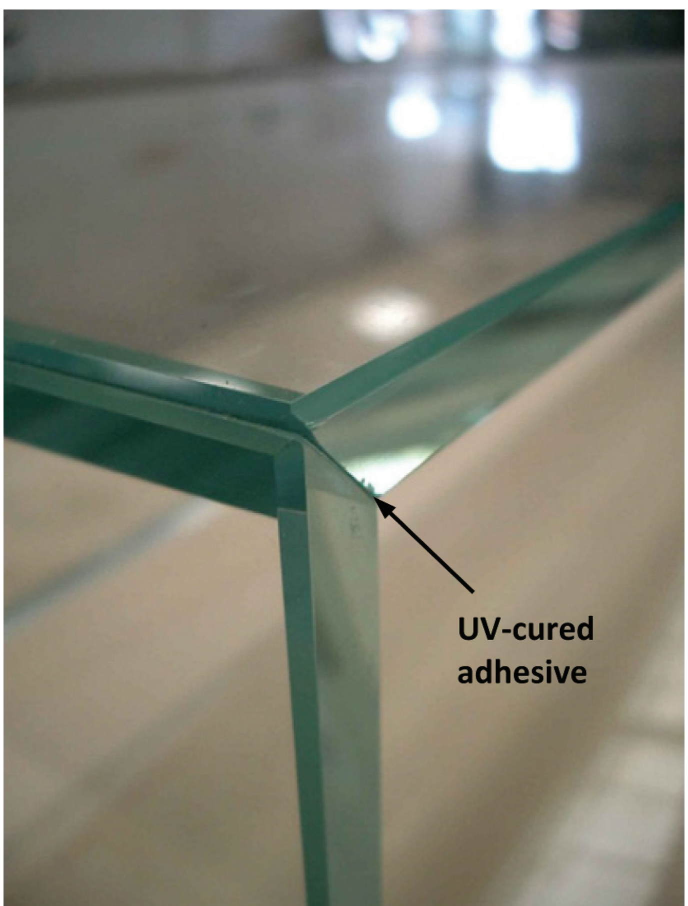

Figure 21.9 UV-cured transparent adhesive joint used in specialist glass furniture application

21.2.2 General performance requirements

The performance requirements that inform the design of structural glass elements often extend beyond the purely structural considerations and include: natural lighting; thermal performance; acoustic considerations; structural performance under normal and exceptional actions; security; durability; and maintenance.

The non-structural performance requirements are beyond the scope of this chapter, but it is very important to note that they will influence and in some cases severely constrain the structural design process (and vice versa). The structural design of glass elements must therefore be carried out with frequent consultation of the other design team members such as the architects, building service engineers, fire consultants, acoustic consultants, etc.

21.2.3 Structural glass design process

The aim of the structural design process is to devise an efficient solution that satisfies the performance criteria. There are two important particularities when using glass as a structural material. Firstly, glass is an inherently brittle material, and the consequences of failure and the residual post-fracture performance must be addressed explicitly during the design process. Secondly, the engineer must have a suitably high degree of confidence in their calculations. However, the desktop design methods and the construction techniques used in glass are still in their infancy, and it is often necessary to perform prototype tests to validate the calculations.

The design of glass members and glass structures will generally require the following steps:

1. Define the performance criteria required for ultimate limit state (ULS) and serviceability limit state (SLS). The ULS performance requirements should be based on a risk analysis of the intended use of glass and the consequences of failure. A simple risk analysis for glass design and the resulting special performance requirements are shown in **Table 21.1**.

Table 21.1 is based on residential, office and public buildings for normal use. It is important to note that the consequences of failure are a function of the type of building. Therefore, the failure of a secondary structural member in a densely occupied building (e.g. a concert hall) might be considered as a 'high consequence' rather than 'medium' as shown in **Table 21.1**. Conversely, the failure of a secondary structural member in a sparsely occupied building (e.g. in an agricultural building) might be reclassified as a 'low consequence'. General guidelines on detailed on risk analysis in buildings are available in BS EN1990 (Haldimann *et al*., 2008) and BS EN1991-1-7:2006.

This risk analysis is not merely a form filling exercise. The consequence of failure for the glass member in question will dictate what special performance measures need to be adopted by the designer as shown in the lower part of **Table 21.1** and will also influence the selection of the design load factors as described in Section 21.4.1.

2. Devise a glass member/structure that meets the performance requirements. This can be done by:

- Rules of thumb and simple calculations for preliminary sizing and to generate alternative options at the early design stage.
- More accurate numerical and analytical design methods during the detailed design stages.
- Prototype testing to validate calculations where there is an unacceptably low confidence in the calculations, for example, where a novel use of glass is being proposed.

Application-specific performance requirements and verification methods for typical uses of glass in buildings are shown in **Table 21.2**. Further details on the specific criteria and the verification methods are given in Section 21.4.

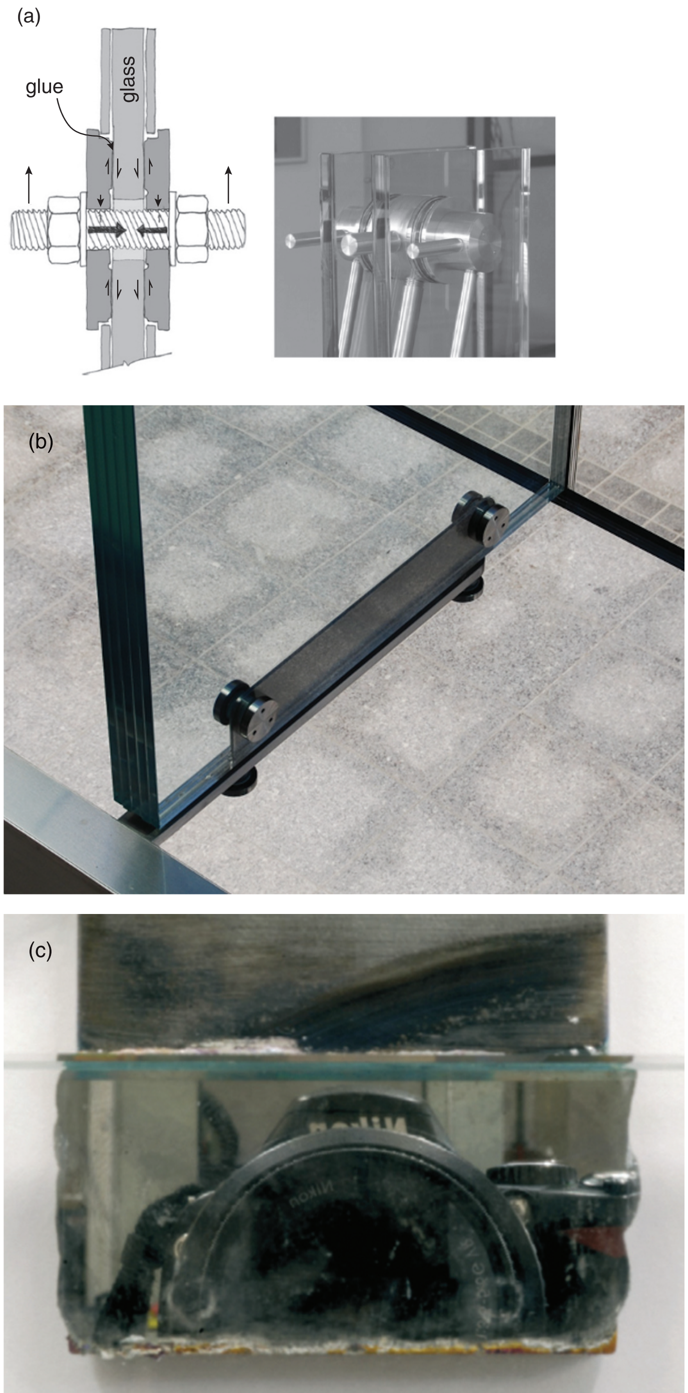

Figure 21.10 Novel/hybrid connections: (a) bonded friction-grip connection (Haldimann *et al*., 2008); (b) stainless steel insert bonded to Sentry Glass Plus interlayer in laminated glass which is in turn bolted to the substructure (image courtesy of Eckersley O'Callaghan Engineers); (c) soldered steel–glass connection

21.3 Materials and mechanical properties

21.3.1 Basic manufacture and mechanical properties

The primary material produced by the float process is flat soda-lime-silica glass. The process consists of melting the raw materials, namely: silicon dioxide (silica); sodium carbonate (soda) and lime to reduce the high melting temperature; and dolomite to increase durability and delay crystallisation. The materials are melted at 1500°C in a furnace and a continuous ribbon of molten glass is fed onto and floats on a bath of molten tin. As the glass cools rapidly from 1100°C to 800°C in the float bath, its viscosity increases and prevents crystallisation, effectively becoming an amorphous isotropic solid.

Due to the absence of slip planes or dislocations glass exhibits almost perfectly elastic, isotropic behaviour and brittle fracture. The theoretical tensile strength (i.e. that derived from intermolecular forces) of annealed glass is exceptionally high and may reach 32 GPa. However, the actual tensile strength is several orders of magnitude lower. The reason for this discrepancy is the presence of stress raising flaws on the surface of the glass, known as Griffith flaws, which arise from manufacturing, handling, weathering and malicious attack. The effect of these flaws on the strength of glass is illustrated in **Figure 21.11**. Other physical properties of glass are easier to establish and are shown **Table 21.3**.

Flaws are unable to propagate in the presence of compression; as a result the compressive strength of glass is much larger than the tensile strength. The compressive strength is, however, irrelevant for structural applications, as transverse tensile stresses arising from Poisson's ratio effects or from buckling tend to cause indirect tensile failures and dominate the design.

	Location	Consequence of potential failure				
		Severe	High	Medium	Low	Very Low
Risk analysis	Primary structure (e.g. load-bearing columns, beams, floors and walls)	Key elements that determine the stability of the structure	Primary structural elements whose failure would result in localised collapse			
	Secondary structure (e.g. glass fins and top hung bolted glass facades)			Secondary structural elements whose failure would be contained within the secondary structure		
	Infill panel (including glass partitions and balustrades)			Glass members in critical locations where failure is limited to the member in question, but there is a potential for injury	Infill panels in non-critical locations	Infill glass elements in locations where people do not normally enter
Special performance requirements		Design as key element with adequate reserve strength and protect occupants from glass fragments	Provide adequate reserve strength and protect occupants from glass fragments OR Provide alternative load paths and protect occupants from glass fragments		none	none

Table 21.1 Simple risk analysis and corresponding special measures

Application	Performance criteria	Verification method
1. Vertical glazing (subtending an angle of <10° to vertical)	Resistance to wind, thermal stresses and altitude	Calculations to satisfy normal use from national codes of practice
2. Vertical glazing (<10° to vertical) subjected to blast and/or hurricane loading	As for (1) + resistance to flying debris from hurricanes	Calculations as for (1) + flying debris test for hurricanes
3. Vertical glazing (<10° to vertical) with a safety barrier/balustrade role	As for (1) + resistance to human static horizontal load + human impact	Calculations as for (1) + calculations for horizontal static load + testing for human impact
4. Inaccessible overhead glazing (subtending an angle of ≥10° to vertical)	Resistance to wind, thermal stresses, altitude, snow and impact if objects can be thrown or dropped onto glass	Calculations for all loads other than impact where hard body impact testing is required
5. Horizontal glazing accessible for maintenance purposes	As for (4) + resistance to static live loads + resistance to maintain personnel falling and dropping tools onto glass	Calculations for all loads as for (4) + testing for hard body impact, soft body impact and post-fracture strength
6. Horizontal glazing accessible to public	As for (4) + resistance to static live loads + resistance to public dropping objects and falling onto glass	Calculations for all loads as for (4) + testing for hard body impact, soft body impact and post-fracture strength
7. Novel uses of glass including novel materials, connections, etc.	Varies, depending on consequence of failure (refer to **Table 21.1**)	Design assisted by testing involving full scale prototype tests (refer to EN 1990 [2])

Table 21.2 Typical performance requirements and verification methods

It is possible to calculate the strength of glass from linear elastic fracture mechanics, but this requires knowledge of the location and size of the critical flaw, which in most structural design applications is unknown. It is, therefore, more convenient to express the strength of glass statistically in terms of:

- surface condition (i.e. severity and distribution of surface flaws);
- surface area exposed to tensile stress;
- surface stress history (i.e. magnitude and duration);
- environmental conditions (especially humidity).

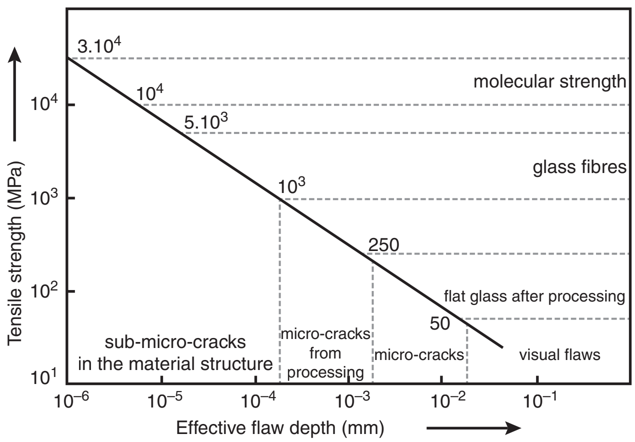

Figure 21.11 Short-term tensile strength as a function of flaw depth (Haldimann *et al.*, 2008)

Density	2500 kgm^{-3}
Young's modulus	70 GPa – 74 GPa
Poisson's ratio	0.22 – 0.24
Fracture toughness	0.78 MPa m$^{1/2}$
Knoop hardness	6 GPa
Annealing point	$10^{13.5}$ Pa s (520°C)
Thermal conductivity	1 W m^{-1} K^{-1}
Coefficient of thermal expansion	7.7×10^{-6} K^{-1} – 9×10^{-6} K^{-1}

Table 21.3 Physical properties of soda-lime-silica glass

21.3.2 Surface condition

A large scatter of strength data is always obtained when a batch of nominally identical test pieces of a glass is broken in a carefully controlled way. This dispersion is a result of the variations in surface flaw characteristics and may be represented by a 2-parameter Weibull distribution.

$$P_f = 1 - \exp\left(-kA\sigma_f^m\right) \tag{21.1}$$

where P_f is the probability of failure, σ_f is the major principal stress at failure, and m and k are two interdependent parameters whose values describe the mean and degree of scatter of the test data. The magnitude of the two parameters m and k are a measure of the variability and the mean strength respectively. There are therefore considerable differences in m and k values between new (as-received) glass and heavily weathered or damaged glass. Typical values are $6.0 \le m \le 9.0$ and 1.32×10^{-69} m^{-2} Pa$^{-9.1}$ $\le k \le 7.19 \times 10^{-45}$ m^{-2} Pa$^{-6.0}$ (Haldimann, 2006; Haldimann *et al.*, 2008).

21.3.2.1 Surface area

The probability of encountering a critical flaw in a glass plate increases with larger surface areas. A large glass plate is therefore statistically weaker than a smaller one. This phenomenon

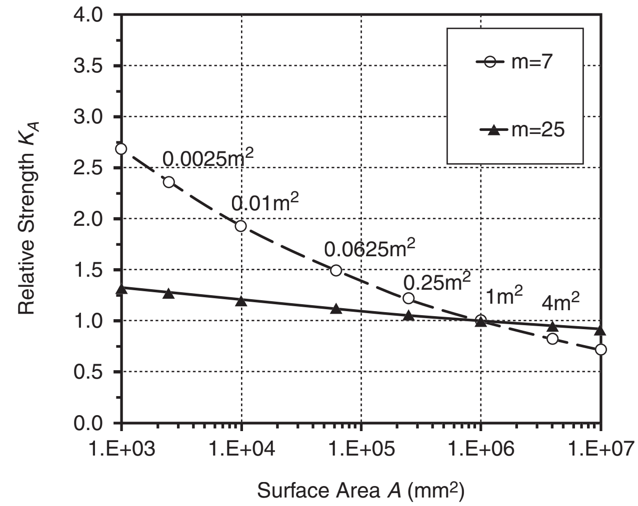

Figure 21.12 Relative strength as a function of surface area

commonly referred to as 'size effect' is represented by the stressed surface area factor K_a:

$$k_A = \frac{P_{f,A}}{P_{f,A0}} = \frac{\sigma_{f,A}}{\sigma_{f,A0}} = \left(\frac{A}{A_0}\right)^{1/m} \tag{21.2}$$

where A is the surface area subjected to the tensile stress and A_0 is the reference surface area. Most glass codes of practice including the draft European standard (prEN 13474-1, 2007) and the American standard (ASTM E1300-09a, 2009) adopt A_0 = 1 m^2. As discussed above, the value of m is typically 7, but it can range from 25 for glass with a very uniform degree of damage to 3 for very random damage. This will decrease or increase the size effect, respectively, as shown in **Figure 21.12**.

21.3.2.2 Stress history and environmental conditions

Fast fracture occurs when the stress intensity at the tip of a flaw exceeds the plane strain fracture toughness resulting in crack growth at approximately 1500 m/s. In the presence of a crack opening stress and humidity, an intermediate state exists wherein the flaws grow sub-critically (at speeds between 0.001 m/s and 1 m/s). This phenomenon, known as stress corrosion (or static fatigue), is relevant to the structural use of glass as it causes a reduction in tensile strength with tensile stress and time. A stress duration factor k_{mod} may be used to describe this reduction in strength, which for constant environmental conditions and a constant stress over a duration t, is:

$$k_{mod} = \frac{P_{f,t}}{P_{f,t0}} = \frac{\sigma_{f,t}}{\sigma_{f,t0}} = \left(\frac{t_0}{t}\right)^{1/n} \tag{21.3}$$

where t is the stress duration and t_0 is the reference stress duration. The static fatigue constant, n, is a function of humidity and is conservatively assumed to be n = 16.

The draft European standard for glass in building (prEN 13474-1, 2007) adopts a reference duration t_0 = 5 seconds, therefore equation (21.3) can be re-written as $k_{mod} = 0.633t^{-1/16}$ where t is the stress duration in hours. **Figure 21.13** shows this stress corrosion with as a function of time. A simple time-step function corresponding to the three stress durations suggested in the European standard (prEN 13474-1, 2007) is superimposed on **Figure 21.13** and described further in **Table 21.4**.

21.3.3 Secondary processing and mechanical properties

21.3.3.1 Toughened glass

Annealed glass can be either chemically or thermally treated to reduce the influence of surface flaws. Thermal toughening is more economical and involves heating the glass to 625ºC followed by rapidly cooling the surfaces. As the inner core of the glass cools and contracts it puts the outer surface into compression. This results in a parabolic residual stress distribution through the thickness h of the glass, where the glass surface is in compression f_{rk} (**Figure 21.14**).

The magnitude of surface pre-compression, f_{rk}, is governed by the rate of cooling and by the proximity of free edges to the point of interest. Two distinct classes of thermally treated glass are available: heat strengthened glass and the stronger fully toughened glass (also known as tempered glass). In the United States, the minimum allowable far-field pre-compression for fully toughened glass is 69 MPa (Overend, 2010), whereas in Europe the minimum far-field pre-compression equates to approximately 90 MPa (BS EN12600:2002). For heat strengthened glass the far-field pre-compression ranges between 24 MPa and 52 MPa.

In toughened glass, surface cracks may only propagate after the surface pre-compression has been overcome. Equation [21.1] may therefore be extended to toughened glass as follows:

$$P_f = 1 - \exp\left(-kA\left(\sigma_f - f_{rk}\right)^m\right) \tag{21.4}$$

where f_{rk} is the residual compressive stress on the glass surface.

21.3.3.2 Laminated glass

Laminated glass consists of two or more glass plates bonded together by a transparent polymer interlayer, normally polyvinyl butyral (PVB). The nominal thickness of a single PVB foil is 0.38 mm and it is normally built up into two layers (0.78 mm) or four layers (1.52 mm). Laminating the glass has no observable effect on the crack propagation, but has a significant influence on the post-fracture behaviour.

PVB is a viscoelastic material and is susceptible to creep. The stiffness of the interlayer and the flexural behaviour of laminated glass are therefore influenced by the magnitude and duration of loading and temperature (**Figure 21.15**). At

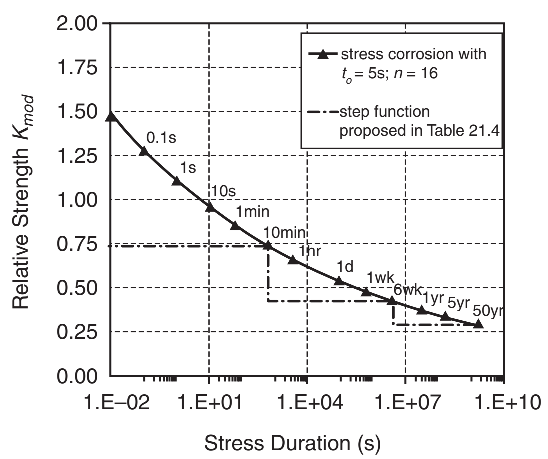

Figure 21.13 Relative tensile strength as a function of stress duration

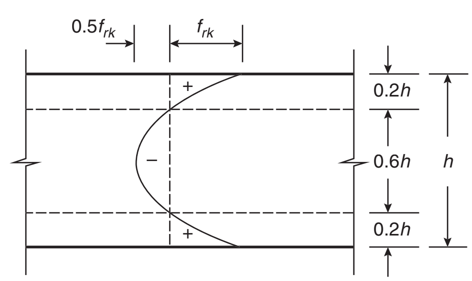

Figure 21.14 Residual stresses in thermally toughened glass

Design combination	Stress duration	Stress duration factor k_{mod}
Long-term combination, F_{dL} e.g. self weight	t_f > 6 weeks	0.29
Medium-term combination, F_{dM} e.g. sustained imposed loads, seasonal temperature, snow and self weight	6 weeks ≥ t_f > 10 minutes	0.43
Short-term combination, F_{dS} e.g. wind, access loads, sustained imposed loads, wind, temperature, snow and self-weight	t_f ≤ 10 minutes	0.74

Table 21.4 Load duration combination proposed by draft European Standard (prEN 13474-1, European Committee for Standardization, 2007)

room temperature, PVB is comparatively soft with an elongation at breakage of more than 200%. At temperatures below 0°C and for short load durations, PVB is sufficiently stiff (G ≈ 1 GPa) and transfers longitudinal shear from one pane of glass to another. At higher temperatures and long load durations, the shear transfer is greatly reduced. It is common practice to assume some degree of shear transfer (≈ 20%) for short-term loading of PVB and to ignore shear transfer for medium- and long-term loading.

Alternative interlayers such as the stiffer and stronger Sentry Glass Plus (SGP) provide enhanced post-fracture performance, SGP is 30–100 times stiffer than PVB and has the ability to absorb 500% the tear energy of PVB. However, the higher stiffness at high strain rates of SGP compared to PVB means that a larger proportion of the incident shock loads will be transmitted to the supporting structure.

In structural design, it is often convenient to express the actual build-up of laminated glass as an equivalent monolithic glass thickness.

The equivalent thickness for calculating the bending deflection is given by:

$$h_{eq,\delta} = \sqrt[3]{(1-\varpi)\sum_i h_i^3 + \varpi\left(\sum_i h_i\right)^3} \qquad (21.5)$$

and the equivalent thickness for calculating the bending stress in the i^{th} ply is given by:

$$h_{eq,\sigma} = \sqrt{\frac{\left(h_{eq,\delta}\right)^3}{\left(h_i + 2\varpi h_{m,i}\right)}} \qquad (21.6)$$

where $0 \le \varpi \le 1$ represents no shear transfer (0) and full shear transfer (1); h_i is the thickness of the i^{th} glass plies; $h_{m,i}$ is the distance between the mid-plane of ply i and the mid-plane of the laminated glass unit, ignoring the thickness of the interlayers (**Figure 21.16**).

21.3.4 Design strength

The design tensile strength of annealed glass f_{gd} is calculated by combining the effects of stress duration, surface area and residual surface pre-stress from the toughening process:

$$f_{gd} = \frac{k_{\text{mod}}\ k_A\ f_{gk}}{\gamma_{mA}} + \frac{f_{rk}}{\gamma_{mV}} \qquad (21.7)$$

where f_{gk} is the characteristic strength of annealed glass (**Table 21.5**) and γ_{mA} is the material safety factor for annealed glass (γ_{mA} = 1.8 for ultimate limit state and γ_{mA} = 1.5 for serviceability limit state), f_{rk} is the characteristic surface prestress induced by the toughening process and γ_{mV} is the material safety factor for toughening process (γ_{mV} = 1.2 for ultimate limit state and γ_{mV} = 1.0 for serviceability limit state).

21.4 Limit state design and loads on glass structures

21.4.1 Actions on glass structures

The actions on glass structures are largely similar to the actions on other construction materials. However, the strength of glass is very sensitive to surface flaws. Therefore, unlike other materials, the complete action history (rather than the extreme value alone) should be considered as it can have a significant effect on the strength of the glass element. The reduction in strength may be caused by (a) actions such as vandalism, wind borne debris, etc., that may cause macroscopic damage to the glass surface and/or (b) by sustained surface tensile stress that causes microscopic sub-critical crack growth (cf. section 21.3.1).

This section provides some further information on glass-specific actions that are summarised in **Table 21.6**.

21.4.1.1 Static imposed loads from normal use

Vertical static imposed loads are those arising from occupancy (refer to EN1991-1-1). Imposed horizontal loads can often be critical in the design of glass parapets, partitions and other glass barriers.

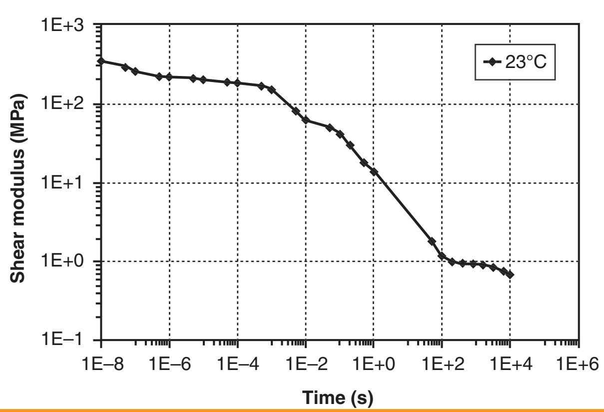

Figure 21.15 Shear modulus G of PVB at various temperatures

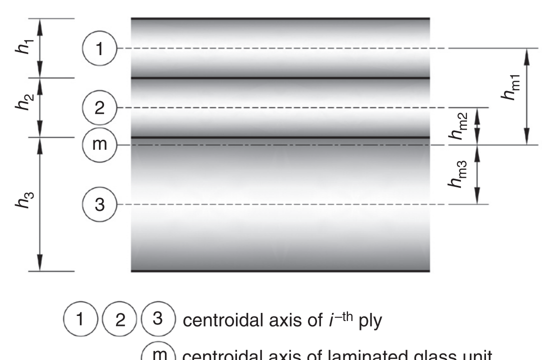

Figure 21.16 Dimensions for equivalent thickness calculations of laminated glass (after prEN 13474-1 (European Committee for Standardization (2007))

Location	Glass type					
	Fully toughened glass		Heat strengthened glass		Annealed glass	
	f_{rk} (MPa)	γ_{MV}	f_{rk} (MPa)	γ_{MV}	f_{rk} (MPa)	γ_{MV}
Far field surface	90	1.2	40	1.2	45[b]	1.8[b]
Edge	67.5[d]	1.2	30[d]	1.2	21[c]	1.8
Hole	58.5[e]	1.2	26[e]	1.2	21[c]	1.8

[a] f_{rk} for $t_f = 5s$
[b] $f_{rk}/\gamma_{MV} = 45/1.8$ corresponds to $P_f = 1/2000$ for new or uniformly weathered glass. A larger value of γ_{MV} should be used for naturally weathered glass and a lower value of f_{rk} should be used for heavily damaged glass.
[c] $f_{rk}/\gamma_{MV} = 21/1.8$ corresponds to ground glass edges with flaws ≤ 1 mm long and ≤ 0.5mm deep. For highly polished glass or as-cut glass, higher / lower values should be used respectively.
[d] corresponding to 75% of far-field surface stress by meeting edge distance recommendations in EN12150:1.
[e] corresponding to 65% of far-field surface stress by meeting edge distance recommendations in EN12150:1.

Table 21.5 Characteristic values of toughened and annealed glass and corresponding material partial factors for ultimate limit state

Action	Guidelines
Self-weight	0.25 kN/m³
Static imposed loads	Vertical static loads to national / international codes (e.g. EN1991-1-1) [11]. Horizontal static load on parapets or partitions ≤ 1 kN/m² applied at height of 1.2m. For buildings susceptible to large crowds consult EN1991-1–1 [11].
Wind load	Net wind pressure calculations based on national / international wind codes (e.g. EN1991-1-4 [12]) for simple / low rise buildings. Wind tunnel testing for buildings with complex geometries / intricate facades.
Internal pressure in IGUs	For stiff panes: $p_{net} = 0.34(T - T_p) + 0.012(H - H_p)$ For flexible panes: As above but with a reduction in p_{net} due to change in volume of the cavity.
Snow load	Snow load and snow drift from national and international codes.
Thermal stress / strain	Provide adequate movement joints for thermal movement between glass and other materials. Maximum ΔT_{adm} within glass: 35K for as cut AN glass h ≤ 12 mm 45K for polished AN glass h ≤ 12 mm 30K for as cut AN glass h ≥ 15 mm 35K for polished AN glass h ≥ 15 mm 30K for HS glass 30K for FT glass
Human impact (including maintenance)	Barriers and partitions: Soft body impact test on vertical barriers and partition performed with 50 kg impactor to EN12600 [9] to meet recommended application-specific classification to national codes (e.g. BS6262-4 [13]).
	Roofs for maintenance access only: Sequence of: Soft Body impact to ACR(M)001 [14]; hard body impact to BS EN 356 [15] and a static load test of 180 kg on the fractured glass for 30 min to assess post-fracture performance.
	Roofs, floors and staircases for public access: Sequence of soft body impact to ACR(M)001 [14]; hard body impact to BS EN356 [15] and a static load test with 50% of the working load on the fractured glass to assess post-fracture performance.
Wind-borne debris	Generally required in hurricane / typhoon-prone regions. Timber missile impact tests to ASTM E1886 [16] and ASTM E1996 [17].
Hail	Not normally required in the UK. Test described in BS EN13583 [18] may be adapted to suit.
Intrusion	Hard body impact test and swinging axe test to BS EN356 [19].
Blast	Preliminary sizing using pressure-impulse charts generally verified by arena blast tests BS EN13541 [19] or GSA 2003 [20].
Movement of substructure	Provide adequate movement joints.

Table 21.6 Summary of typical loads and actions on glass structures

Detailed guidelines on loading is given in (EN1991-1-1), but generally consists of a line load no greater than 1 kN/m applied at no higher than 1.2 m from the finished floor level.

21.4.1.2 Wind

Wind induced pressures on the building envelope are a function of the mean wind velocity and the turbulence intensity. National and international wind loading codes of practice (e.g. EN1991-1-4) are available, but they are limited to simple building geometries and offer limited guidance on complex geometries or intricate facades. As a result, wind pressures obtained from these codes are often supported with wind tunnel testing when the building has an unusual geometry. Furthermore load amplification can occur when the natural frequency of the glass structure is less than 1 Hz (e.g. large span and/or slender facades).

BS 6262-3 provides an abbreviated method for determining the design wind pressure on glass facade panels and includes a series of design charts for sizing rectangular glass plates with simple supports along the four edges.

The draft European standard for Glass in Buildings (pr EN 13474) recommends that the stress corrosion caused by repeated wind loading equates to the gust design pressure applied as a 10 min static pressure on the glass. This was found to be safe, but in some cases overly conservative (Zammit *et al.*, 2008). Accurate mathematical predictions of the sub-critical crack growth (and hence the strength) of a glass panel subjected to real-world wind pressures is not a trivial task and is the subject of ongoing research.

21.4.1.3 Internal pressure in insulating glazing units (IGUs)

A pressure difference, also known as the isochore pressure, between the sealed cavity of an IGU and the environment will arise when there is either a difference in altitude and/or a difference in temperature between the place of production and the place of installation of the IGU.

Assuming a constant volume, the net pressure p_{net} in kPA is given by:

$$p_{net} = 0.34\left(T - T_p\right) + 0.012\left(H - H_p\right) \qquad (21.8)$$

where T_p and H_p are the cavity temperature in Kelvin and the altitude in metres at the place of production and T and H are the cavity temperature in Kelvin and the altitude in metres at the place of installation.

In reality the cavity volume changes as the glass panes deform under pressure. The isochore pressure is therefore reduced by the flexibility of the panes. Guidelines for calculating this effect in rectangular double glazed units is given in pr EN 13474 (Haldimann *et al.*, 2008).

21.4.1.4 Thermal stress

Thermal stresses arise from:

(a) Diurnal and seasonal temperature variations leading to differential movement between the glass and its sub-frame. These stresses are normally prevented altogether by providing adequate movement joints as shown in the point-supported glazing panel in **Figure 21.17**.

And/or

(b) Thermal gradients across the glass surface generally resulting from different exposures to solar radiation, for example, the solar energy absorbed by the unshaded central regions of a facade panel will be significantly higher that the energy absorbed by the shaded edges of that panel. The temperature gradient is a function of the absorption coefficient of the glass, the incident radiation, the glass emissivity and the ambient temperature. The French code [23] provides guidelines on how to calculate the thermal gradients which must not exceed the allowable maximum temperature difference shown in **Table 4.1**. Further guidelines are available in a technical note by CWCT [24].

21.4.1.5 Human impact on vertical glazing

This is a key requirement for barriers and partitions and is intended to minimise injury caused by persons falling through the gaps in barriers and/or by contact with glass fragments. The test involves a soft body pendulum test. The imapactor consists of two rubber tyres wrapped around 50 kg steel cylindrical impactor. The test described in EN12600 is used to classify the impact resistance by observing the mode of breakage. Other national codes of practice (e.g. BS 6262-4) must be consulted for the recommended classification for particular applications/locations.

21.4.1.6 Impact from hail and windborne debris

These are not normally specified in the UK, but are required in locations such as Florida (wind borne debris) and the Alps (hail resistance). ASTM E886 and ASTM E1996 describe tests for firing timber missiles at glazing panels to simulate flying debris. BS EN13583 describes tests for assessing the hail resistance of flexible roofing materials, and can be adapted to

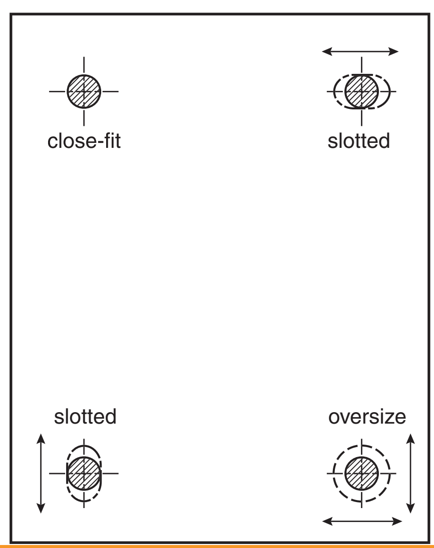

Figure 21.17 Provision for movement in point supported glazing panel (Haldimann *et al.*, 2008)

glazing, but the energy imparted on impact will depend on the angle that the glass subtends to the vertical.

21.4.1.7 Intrusion

The degree of security provided by glazing is tested in terms of the resistance to manual attack. Two tests are described in BS EN356: (a) hard body drop tests consisting of a 100 mm diameter steel sphere dropped onto a horizontal glass panel from heights ranging from 1.5 m to 9 m; (b) mechanised swinging axe tests on a vertical glass panel. In both tests the glazing is classified in terms of its ability of the glass to resist penetration by the impactor.

21.4.1.8 Maintenance-related impact on horizontal glazing

The maintenance loads on glazed roofs are a function of the degree of access afforded to maintenance personnel. This ranges from glazed roofs that can be walked on for occasional maintenance to roofs onto which people are physically prevented from walking, falling or dropping tools. The procedure therefore consists of a sequence of tests involving a soft body impact test with a 45 kg impactor dropped from a 1.2 m height onto the glass panel to simulate a person falling onto the roof from a standing position [13]; a hard body impact test with a 100 mm diameter steel sphere dropped onto a horizontal glass panel from a 1.2 m height to simulate the effect of tools being dropped on the roof [15]; a 180 kg static load for a duration of 30 minutes with all plies broken to simulate one person falling onto the glass and becoming injured and another person giving assistance. The test is successful if the glass unit does not fall out, the impact body does not penetrate the glass and no dangerous glass fragments fall out [25]. In addition, roofs that can be walked on by maintenance personnel will require adequate slip resistance, which in some instances can be provided by special footwear.

21.4.1.9 Public access impact on horizontal glazing

In addition to the standard superimposed live loads specified in EN1991-1-1, glazing with unrestricted access to foot traffic such as glass floors and glass staircases must have an adequate impact and post-fracture capacity. Hard body and soft body impact tests similar to those described for maintenance loads (Section 21.4.1.8) should be performed, but the glass pane should be preloaded to half the working load for the impact tests. The post-fracture capacity is assessed by maintaining half the working load on the glass unit with all plies broken for a duration of 30 minutes. It is also advisable to ensure that the unloaded unit with all plies broken does not collapse within 24 hours.

21.4.1.10 Blast

The shock wave from an explosion creates a positive high pressure at its leading edge known as the positive phase that decays rapidly to ambient pressure; this is followed by a much longer negative pressure known as the suction phase. The latter does not normally lead to major structural damage, but often causes widespread damage to light cladding and glazing. The response of glazing in blast loading is generally assessed by means of dynamic analysis and verified by mounting the glass onto a test cubicle and performing a blast test in a secure range testing site. The aim of these tests is to determine the hazard levels by measuring how far glass fragments are projected into the test cubicle. Alternatively testing can take place in shock tubes. Pressure impulse charts are useful for initial sizing of simply supported laminated glass panels, but novel boundary conditions require testing. Further guidelines on testing for blast resistance are available in BS EN 13541 and GSA 2003.

21.4.2 Limit state design with glass

As with other materials, glass structures must be designed and constructed to minimise injury and loss of property/business. This is achieved by satisfying the following limit states:

- Ultimate limit state for normal use: adequate strength and stability for service life loads including superimposed live loads, loads arising from temperature variations, loads during the construction stage, and maintenance loads.
- Serviceability limit state for normal use: limiting deflections and vibrations to ensure adequate functioning and appearance of the structure including comfort of users.
- Ultimate limit state for exceptional actions: limit and/or delay the damage to a structure from the exceptional actions such as fire, impact, blast and earthquakes. Any damage in the structure shall not be disproportionate to the cause.

21.4.2.1 Action combinations for ULS (normal use)

The design loads F_d for ULS arising from normal use may be determined from:

$$F_d = \gamma_G G \text{ "+" } \gamma_Q Q_{k,1} \text{ "+" } \gamma_Q \sum_i \psi_{0,i} Q_{k,i} \qquad (21.9)$$

where γ_G is the partial factor for permanent actions, G is the value of permanent actions (e.g. self-weight load, permanent equipment), γ_Q is the partial factor for variable actions, $Q_{k,1}$ is the characteristic value of the leading variable action (e.g. imposed load on floor, wind, snow) and $\psi_{0,i}$ is the combination factor for accompanying variable actions.

This includes self-weight, static imposed, wind, snow and temperature. It excludes impact and blast loads as well as combinations that involve simultaneous static and impact loads e.g. maintenance impact (cf. Section 21.4.1.8) and public access impact (cf. Section 21.4.1.9) that are assessed separately.

There are no statistics to predict the likelihood of horizontal static (barrier) loads and wind loads occurring simultaneously. For buildings where people may congregate it is common practice to consider the combined action of full wind load plus half the horizontal barrier load or half the wind load plus the full horizontal barrier load whichever is greater. For other buildings (e.g. residential and office buildings) the wind load and the barrier load should be considered separately.

The strength of annealed glass is very sensitive to stress history (cf. section 21.3.2). It is therefore sensible to assemble three fundamental combinations:

- The worst combination of actions F_{dS} that is expected to occur within any time period t_s during the service life of the structure, where $0 < t_S \leq 10$ min resulting in a major principal surfaces stress $\sigma_{I,S}$.
- The worst combination of actions F_{dM} that is expected to occur within any time period t_s during the service life of the structure, where 10 min $< t_M \leq 6$ weeks resulting in a major principal surfaces stress $\sigma_{I,M}$.
- The worst combination of actions F_{dL} that is expected to occur within any time period t_s during the service life of the structure, where 6 weeks $< t_L \leq 50$ years resulting in a major principal surfaces stress $\sigma_{I,L}$.

Table 21.7 shows a summary of the recommended partial load factors for normal use. The reduction of partial factors from left to right reflects the fact that failure of secondary glass structures and infill panels do not have the same human and/or economic consequence as the failure of main structures. The consequence of failure is determined by a risk analysis of the glass member being designed as described in Section 21.2.3.

The short-, medium- and long-term principal stresses $\sigma_{I,S}$,$\sigma_{I,M}$ and $\sigma_{I,L}$ resulting from these loads may then be compared with the corresponding time-resolved design strengths $f_{gd,S}$, $f_{gd,M}$ and $f_{gd,L}$ obtained from equation (21.7) by means of the stress–history interaction equation:

$$\frac{\sigma_{1,S}}{f_{gd,S}} + \frac{\sigma_{1,M}}{f_{gd,M}} + \frac{\sigma_{1,L}}{f_{gd,L}} \leq 1 \qquad (21.10)$$

The interaction equation accounts for the fact that the strength of glass at any point in its service life is a function of the stress corrosion (sub-critical crack growth) caused by the preceding actions.

This approach is inherently conservative as it assumes that the entire surface of the member being designed is subjected to the maximum principal stress, thereby overestimating the probability of failure. A more accurate approach is to consider a close-to-reality state of stress and to summate the contribution of these stresses to the probability of failure. In this more accurate approach, the numerators in equation (21.10) represent the equivalent uniform state of stress on the glass member for short-, medium- and long-term loads [5].

21.4.2.2 Action combinations for SLS

The design loads for SLS arising from normal use may be determined from:

$$F_d = G \text{ "+" } \psi_1 . Q_{k,1} \text{ "+" } \sum_i \psi_{2,i} Q_{k,i} \qquad (21.11)$$

where ψ_1 is the factor for frequent value of a variable action and $\psi_{2,i}$ is the factor for quasi-permanent value of a variable action. Recommended values for the partial load factors ψ_1 and ψ_2 are shown in **Table 21.7**.

			Consequence of potential failure		
			Severe – High	Medium – Low	Very Low
	γ_G		1.35(1.0)	1.2(1.0)	1.0(1.0)
	γ_Q		1.5 (0)	1.3 (0)	1.1 (0)
Service-life partial action anc combination factors	ψ_0	live†	0.7	0.7	0.7
		wind‡	0.6	0.6	0.6
		snow*	0.6	0.6	0.6
		temperature	0.6	0.6	0.6
	ψ_1	live†	0.7	0.7	0.7
		wind‡	0.9	0.9	0.8
		snow*	1.0	1.0	1.0
		temperature	0.5	0.5	0.5
	ψ_2	live†	0.6	0.6	0.6
		wind‡	0.2	0.2	0.2
		snow*	0.2	0.2	0.2
		temperature	0	0	0

† Shopping and congested areas. For other building categories refer to EN1990.
‡ Based on a mean return period of 50 years. For other probabilities refer to EN1991-1.4.
() Partial factor for favourable action.
* CEN member states except Finland, Iceland, Norway, Sweden and H s 1000m a.s.l.

Table 21.7 Partial load factors

21.4.2.3 Considerations for exceptional actions

Exceptional actions are normally considered separately by performing action-specific tests, as described in Sections 21.4.1.5 to 21.4.1.10.

In the absence of specific test recommendations the design loads for exceptional/accidental actions may be determined from:

$$F_d = G \text{ "+" } A_d \text{ "+" } \psi_1 Q_{k,1} \text{ "+" } \sum_i \psi_{2,i} Q_{k,i} \qquad (21.12)$$

where A_d is the design value of an accidental action.

In glass structures an adequate post-fracture performance, i.e. subsequent to the exceptional event, is often required. The residual post-fracture capacity may be determined by subjecting the fractured glass to a post-breakage design load of $F_d - A_d$ in equation (21.12).

21.5 Practical design recommendations

This section provides guidelines on the use of approximate methods for the design of structural glass. These are not a replacement for detailed calculations, but they can be very useful at early stage design or as a quick check for more detailed numerical analysis. This section also contains references and recommendations for more detailed analysis.

21.5.1 Rules of thumb

Tables 21.8 and **21.9** show approximate span-to-thickness ratios for laterally loaded glass plates and allowable stresses for preliminary design respectively.

	Maximum span / thickness	
Glass type	Vertical	Sloping or horizontal
Annealed glass	150	100
Fully tempered glass	200	150
Laminated annealed glass	150	100
Laminated tempered glass	150	100

Table 21.8 Typical span/thickness ratios

21.5.2 Stability

Glass is predominantly manufactured in thin flat sheets and buckling instability is therefore a key consideration when glass is in compression. Buckling theory is well established and used extensively in the structural design of other materials, but when transferring some of this established theory to glass, the engineer must be aware of: (a) the manufacturing tolerance and initial imperfections in glass and (b) the visco-elastic and temperature dependent behaviour of the interlayers used in laminated glass.

21.5.2.1 Column buckling

The load-carrying behaviour of a monolithic glass column with effective length L_e, cross-sectional area A, initial imperfection w_0 and axial compression N applied at an eccentricity e can be derived directly from the second order differential equation.

The elastic critical (Euler) buckling load is:

$$N_{cr} = \frac{\pi^2 EI}{L_e^{\,2}} \qquad (21.13)$$

The maximum deflection at mid span is:

$$\delta_{\max} = \frac{e}{\cos\left(L_e / 2\sqrt{N/N_{cr}}\right)} + \frac{\delta_0}{1 - N/N_{cr}} \qquad (21.14)$$

and the maximum surface stress is:

$$\sigma_{\max} = \frac{N}{A} \pm \frac{N}{Z}\left(\delta_{\max} + \delta_0 + e\right) \qquad (21.15)$$

where Z is the elastic section modulus about which buckling will occur.

As described in Section 21.3.4, the initial imperfections in annealed glass are very small (typically $\delta_0 \leq$ L/2500), but the thermal toughening process causes roller wave distortions and an overall bow in glass with a combined $\delta_0 \leq$ L/200.

In laminated glass columns, the interlayer provides a shear connection between the glass plates. The complex time and temperature relationship may be simplified by using elastic sandwich theory and by considering the interlayer as a

	Approximate Strength f_{agd}			
Stress and load type	Annealed glass		Fully toughened glass‡	
	Far-field (MPa)	Edge or Hole (MPa)	Far-field (MPa)	Edge or Hole (MPa)
Short-term (e.g. wind action)	18.5	8.5†	93	57†
Medium-term (e.g. snow load, human traffic)	10.5	5†	85	52.5†
Long-term (e.g. self weight, superimposed dead)	7	3†	81	50†

† With ground glass edges (flaws ≤ 1mm long and ≤ 0.5mm deep). For highly polished glass or as-cut glass, higher/lower values should be used respectively.
‡ Tempered glass complying to BS EN12600; f_{agd} shown includes contribution from inherent strength of annealed glass.

Table 21.9 Approximate design strength

perfectly elastic material with a constant shear modulus for a given temperature and stress duration. Equations for critical load, maximum deflection and maximum surface stress are available in Haldimann *et al.* (2008). For long-term loads and/or high temperature environments, it is sensible to ignore the contribution of the interlayer altogether. The initial imperfections in laminated glass are the same as those for the constituent glass plates.

21.5.2.2 Lateral torsional and local buckling

Slender glass members such as fins subjected to bending about their major axis are particularly susceptible to lateral torsional buckling and local buckling (**Figure 21.18**).

For rectangular fins with a cross-section width b and depth d subjected to pure bending M_x, with torsional restraints M_z located l_{ey} apart: the critical elastic bending moment is given by:

$$M_{cr,LT} = \frac{\pi h^3 d}{6 l_{ey}} \sqrt{EG\left(1 - 0.63\frac{h}{d}\right)} \tag{21.16}$$

For fins with torsional restraints M_z and rotational restraints M_y located l_{ey} apart, the critical elastic bending moment is given by:

$$M_{cr,LT} = \frac{\pi h^3 d}{3 l_{ey}} \sqrt{EG\left(1 - 0.63\frac{h}{d}\right)} \tag{21.17}$$

Guidelines for fins subjected to non-uniform bending moments are provided in AS 1288.

Local buckling often governs the sizing of a glass fin; this can be determined approximately from:

$$M_{cr} = \frac{Eh^3}{6(1+\nu)} \tag{21.18}$$

Equations (21.16), (21.17) and (21.18) ignore initial imperfections and the presence of the PVB interlayer that are normally encountered in glass fins. They should therefore be regarded as approximate and used for preliminary design purposes only. Further details on the performance of laminated glass under compressive loads are provided in Haldimann *et al.* (2008).

21.5.2.3 Connections

The inherent brittle nature of glass means that the load-bearing capacity of glass members is often governed by the stress concentrations at the connections particularly when point connections, such as the frameless glazing connections like those described in Section 21. 2.1.3, are used.

Bolted connections

Determining the state of stress around bolt holes is analytically complex and often involves nonlinear finite element analysis. Guidelines on this are beyond the scope of this chapter, but it is important to model the connection as faithfully as possible by, for example, (a) introducing contact elements around the bolt hole to simulate the bearing of the bolt; (b) assigning the appropriate characteristics to the various materials used in the connection; and (c) specifying the correct boundary conditions, particularly where semi-rigid restraints are present.

Given that there is a sufficient end distance c, edge distance $(d - H)/2$, and an adequate intermediate liner is placed between the steel bolt and the glass to reduce hard spots, the strength of bolted connection is governed by the tensile stresses generated by the elongation of the hole. This peak stress occurs at the rim of the hole approximately perpendicular to the direction of the force. Stress concentration factor charts such as those provided in Pilkey (1997) are useful for determining the peak tensile stresses around the hole, particularly at early design stages.

The peak tensile stress concentration K_t around a bolt hole may also be determined from empirical formulae such as that provided by Duerr (1986). For a loading configuration shown in **Figure 21.19** this is given by:

$$K_t = 1.5 + 1.25\left(\frac{H}{d} - 1\right) - 0.0675\left(\frac{H}{d} - 1\right)^2 \tag{21.19}$$

where the stress concentration factor K_t is defined as:

$$K_t = \frac{\sigma_{max}(H-d)t}{P} \tag{21.20}$$

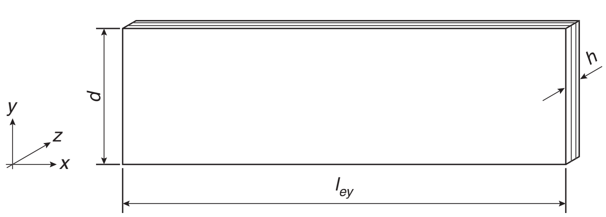

Figure 21.18 Sign convention for lateral torsional buckling

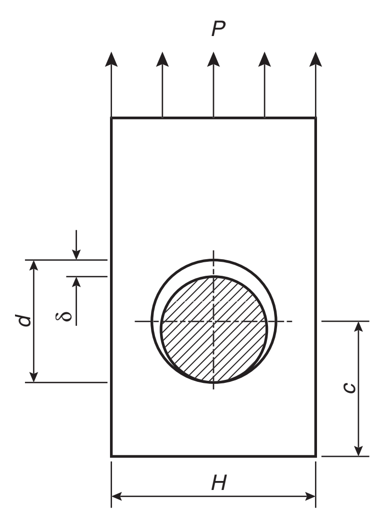

Figure 21.19 Pin and lug notation

For the case of a hole in a plate subjected to a uniaxial stress field, i.e. no load applied through the hole, a useful empirical formula provided by Heywood (1952) is:

$$K_t = 2.0 + \left(1 - \frac{d}{H}\right)^3 \tag{21.21}$$

The state of stress around a bolt hole in glass can in most cases be represented by the superposition of the two standard cases expressed in Equations (21.19) and (21.21). Other standard cases such shear and compression are provided in Fay (2001).

Adhesive connections

Adhesive connections provide the opportunity to distribute the loads onto a large surface area of the glass thereby reducing stress concentrations. In addition, unlike bolted connections, adhesive connections do not require flaw-inducing surface preparation such as hole drilling, and therefore do not adversely affect the strength of the glass in the vicinity of the connection. One major disadvantage of adhesive connections, however, is that the long-term performance of several adhesives is as yet unproven. They should, therefore, be used with caution, particularly where long-term loads or high temperatures are present.

Short-term tensile strength	0.14 MPa
Long-term tensile strength	0.014 MPa
Short-term shear strength	0.07–0.128 MPa
Long-term shear strength	0.007–0.011 MPa
Short-term Young's modulus	1.0–1.25 MPa
Long-term Young's modulus	0.9 MPa
Poisson's ratio	0.49

Table 21.10 Safe design values for structural silicones

Structural silicone, the adhesive used in structural silicone glazing (cf. Section 21.2.1.3), is an electrometric adhesive that has been used for more than 30 years in the facade industry. Its use is regulated by standardised tests [31]. Typical design properties for structural silicone adhesives are shown in **Table 21.10**.

The low stiffness and low strength of structural silicones is generally unsuitable for point connections. There are a number of adhesives that have a suitably high strength and stiffness, but their long-term performance is the subject of ongoing research. A useful summary of properties is shown in **Table 21.11**. It is important to note that the values in this table are based on experimental tests involving short-term loads and cannot be used directly in design.

The data in **Table 21.11** highlight some very important considerations when using adhesives, namely:

- Adhesives are visco-elastic. As a result the shear (and Young's) moduli of the adhesives consist of two components: (1) The visco-elastic component and (2) the residual component. The latter is independent of load duration, but the former decays to zero with stress duration. The apparent stiffness, and the resulting stress concentrations, will therefore be higher for short-term loads such as wind loads, impact and blast. In some adhesives such as epoxies the short-term shear modulus is more than 700% the long-term shear modulus.
- The stronger adhesives, particularly the acrylics, are sufficiently strong and stiff that on loading failure of the joint is governed by the failure of the tempered glass in the vicinity of the adhesive rather than failure of the adhesive.
- The stronger adhesives tend to be less ductile than the weaker adhesives.

An arguably more reliable alternative to the above-mentioned pot adhesives is to use SGP interlayer to bond steel-to-glass or glass-to-glass as shown in **Figure 21.10(b)**. This interlayer (cf. Section 21.3.2.2) is sufficiently stiff and strong to provide a discrete load-bearing connection, but the assembly process is more demanding than pot adhesives as the bonding must be performed in an autoclave.

	Mean Shear Strength[a] (MPa)	Visco-elastic Shear Modulus G_v (MPa)	Residual Shear Modulus G: (MPa)	Mean Pull-Out Strength [c] (MPa)	Ductility	Ease of preparation and tooling	Strength Variability[d]
silicone	0.58	0.031	0.55	1.07	high	med.	low
polyurethane	0.97	1.50	2.09	0.98	high	low	high
epoxy	7.21	201.88	32. 10	3.57	med.	high	med.
2P-acryclic	15.30[b]	195.89	161.00	7.47	low	high	low
UV-acryclic	9.83[b]	386.23	347.81	10.56	v. low	med.	med.

[a] based on short-term loading and equivalent constant shear stress along the 26 mm long single-lap shear joint.
[b] governed by glass failure.
[c] based on short-term loading and equivalent constant tensile stress across the T-peel joint.
[d] based on single-lap shear and T-peel joints.

Table 21.11 Short-term mechanical properties of adhesives (Overend, Jin and Watson, 2011)

21.6 Conclusions

This chapter provided guidelines for the structural design of glass elements in and around buildings, ranging from simplified calculations useful for the primary design stages to a unified design method that incorporates performance requirements, risk analysis and detailed calculations.

In performing these structural calculations, it is important to note that prototype testing is often required when dealing with novel structural glass applications or when assessing post-fracture performance. Furthermore, the structural design of glass is often closely interlinked with other requirements such as acoustic performance, thermal performance and architectural requirements. It is, therefore, essential to undertake the structural design process in close collaboration with the other relevant disciplines.

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21.7.2 Further reading

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Rice, P. and Dutton, H. (1995). *Structural Glass*. London: Taylor & Francis.

Wigginton, M. (2002). *Glass in Architecture*. London: Phaidon.

Wurm, J. (2007). *Glass Structures: Design and Construction of Self-supporting Skins*. Basel: Birkhäuser.

21.7.3 Useful websites

Centre for Window and Cladding Technology, University of Bath – www.bath.ac.uk/cwct

European Research Network on Structural Glass – www.glassnet-work.org

Glassfiles. Online resource including database of conference papers – www.glassfiles.com

Glass and Facade Technology Research Group, University of Cambridge – www.gft.eu.com

Institution of Structural Engineers study group on facade engineering and structural glass – www.istructe.org/technical/study_groups.asp?CID=1013

Society of Glass Technology – www.sgt.org

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