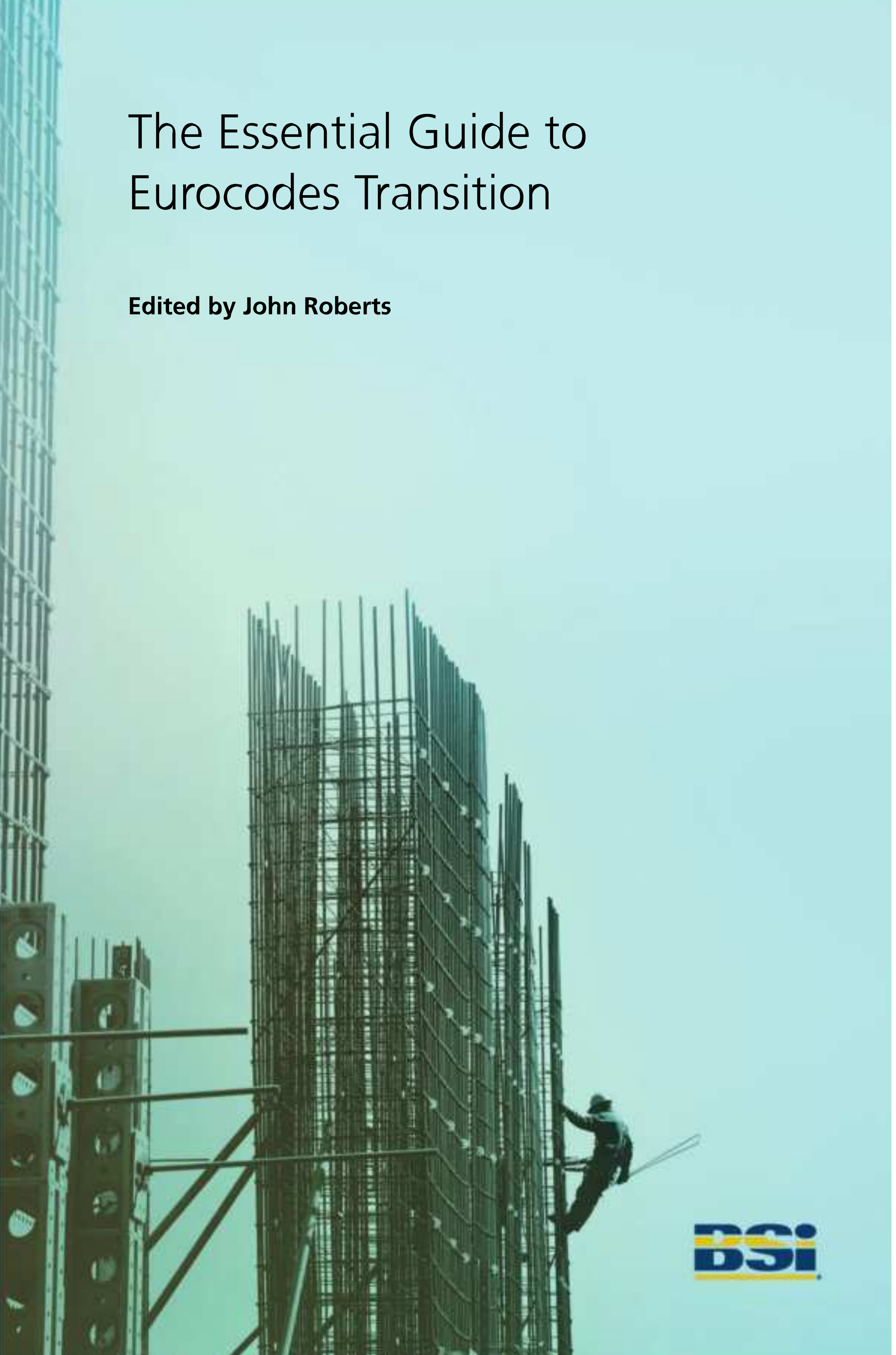
The Essential Guide to Eurocodes Transition

Edited by John Roberts



The essential guide to Eurocodes transition

The essential guide to Eurocodes transition

Edited by John Roberts

British Standards

First published in the UK in 2010

by
BSI
389 Chiswick High Road
London W4 4AL

Typeset in Sabon and Helvetica Neue by Helius – www.helius.biz
Printed in Great Britain by Berforts. www.berforts.co.uk

British Library Cataloguing in Publication Data
A catalogue record for this book is available from the British Library

ISBN 978-0-580-69451-6

Contents

Foreword

Professor John Roberts, Principal, Technical Innovation Consultancy

Welcome to the BSI *The essential guide to Eurocodes transition* Publication prepared to support the UK construction industry through one of the most significant developments in construction standardization. The withdrawal of conflicting national standards at the end of March 2010 presents the opportunity for designers to fully engage with the coherent set of modern design codes which the Eurocodes provide.

Structural Eurocodes are seen as leading the way in structural codes worldwide. Their flexibility enables adoption and use not only within Europe, but internationally. This feature has been recognized by several countries outside Europe and they are already committed to adopting Eurocodes.

The primary objectives of the Eurocodes are to:

- provide common design criteria and methods of meeting necessary requirements for mechanical resistance, stability and resistance to fire, including aspects of durability and economy;
- provide a common understanding regarding the design of structures between owners, operators and users, designers, contractors and manufacturers of construction products;
- facilitate the marketing and use of structural components and kits in EU Member States;
- facilitate the marketing and use of materials and constituent products, the properties of which enter into design calculations;
- be a common basis for research and development, in the construction industry
- allow the preparation of common design aids and software;
- increase the competitiveness of the European civil engineering firms, contractors, designers and product manufacturers in their global activities.

It is a legal requirement from March 2010 that all European public-sector clients base their planning and building control applications on structural designs that meet the requirements of the Eurocodes. In anticipation of this, changes are necessary to the Building Regulations.

Approved Document A for Building Regulations in England and Wales, which provides guidance on how to comply with Part A (structure) of the regulations, lists 22 of the national codes being withdrawn in 2010 but will not be revised by the Communities and Local Government (CLG) department until 2013. CLG have clarified the legal position through a circular letter dated the 29th January 2010 and available on their website http://www.communities.gov.uk/corporate/publications/all/.

The Scottish structural guidance is provided in section 1 of the Domestic Handbook and section 1 of the Non-Domestic Handbook. The Scottish Government plans to publish revised guidance incorporating Eurocodes that will come into effect in 2010.

In Northern Ireland, Technical Booklet D:1994, Structure will be revised to include references to Eurocodes alongside withdrawn British Standards.

The withdrawn British Standards may still be used to achieve compliance with UK building regulations for private sector work but they will no longer be maintained by BSI and will increasingly become out of date.

Each of the Eurocode parts is produced by a subcommittee under the guidance and co-ordination of a technical committee (CEN/TC 250). Delegates of the 29 Comité Européen de Normalisation (CEN) members are represented on CEN/TC 250 and its subcommittees.

Drafts of the Eurocode parts are elaborated by project teams, which are selected by the appropriate sub-committees. A project team consists of about six experts who represent the subcommittee. A vast majority of the project teams include a UK-based expert.

A Eurocode is subject to extensive consultation before it is adopted. Progressive drafts are discussed and commented on by CEN members and their appointed experts. A Eurocode part is adopted only after a positive vote by CEN Members.

This BSI Structural Eurocodes Transition Publication contains articles from leading academics and professionals to help you gain an understanding of the nature of the new codes and to ease your transition into using the new structural design codes.

Structural Eurocodes – Frequently Asked Questions

1 What are Eurocodes?

Structural Eurocodes are a set of harmonized European standards for the design of buildings and civil engineering structures. There are 10 Eurocodes made up of 58 parts that will be adopted in all EU Member states.

In the UK, they will replace over 50 existing British Standards that are due to be withdrawn on 31 March 2010 when full implementation of the Eurocodes will take place.

Eurocodes are a recommended means of giving a presumption of conformity to the essential requirements of the Construction Products Directive for products that bear CE Marking, as well as the preferred reference for technical specifications in public contracts.

Eurocodes cover the basis of structural design, actions on structures, the design of concrete, steel, composite steel and concrete, timber, masonry and aluminium structures, geotechnical design and the design of structures for earthquake resistance.

2 How do I use Eurocodes?

Eurocodes are designed to be used as a suite of documents, which means that for most projects more than one code will be needed e.g. BS EN 1990 Basis of Structural Design is always required.

In addition, Eurocodes are designed to be used with a national annex, which is available separately but is essential for compliance with the code.

Other documents required for using Eurocodes are the so-called Non-Contradictory Complementary Information (NCCI) documents. The status of these documents can vary. As the name suggests they provide supplementary material, that may be useful, but are not always essential for compliance with the Eurocodes.

Other documents include Execution Standards, which provide requirements for execution of structures that have been designed in accordance with Eurocodes.

3 What are national annexes and how do I use them?

In order to allow for the variety of climatic and other factors across the European Union each Member State may produce a national annex for each of the 58 Eurocode parts.

This will include

- Alternative values
- Country specific data (geographical, climatic, etc.)
- Alternative procedures.

It may also contain:

- Decisions on the application of informative annexes
- References to Non-Contradictory Complementary Information (NCCI).

Where a national annex is published it is essential to use it to comply with the Eurocode.

Where no national annex is available or no Nationally Determined Parameters (NDPs) are chosen the choice of the relevant values (e.g. the recommended value), classes or alternative method will be the responsibility of the designer, taking into account the conditions of the project and the National provisions.

NOTE: there will be no national annex to BS EN 1998-3 in the UK.

For information and to purchase national annexes applicable outside the UK contact BSI Distributor sales on 020 8996 7511 or email Distributor.Sales@bsigroup.com.

4 What are Nationally Determined Parameters?

The foreword of each Eurocode states that it recognizes the responsibilities of regulatory authorities in each Member State and protects their right to determine values related to regulatory safety matters at a national level where these continue to vary from State to State.

Accordingly, each Eurocode contains a number of parameters which are left open for national choice, called Nationally Determined Parameters (NDPs). The NDPs account for possible differences in geographical or climatic conditions, or in ways of life, as well as different levels of protection that may prevail at national, regional or local level. Recommended values for the NDPs are also provided in the Eurocodes.

5 What are NCCI and how do I use them?

Non-Contradictory Complementary Information (NCCI) are documents that the National committees consider useful for assisting the user to apply the Eurocode. They are not essential for compliance with the Eurocode but may provide background material or other guidance.

They have been approved by the BSI Committee and are usually listed in Clause NA.4 of the national annex.

This does not mean that all NCCI documents are produced by BSI however. They are not necessarily British Standards and may be published by other organisations.

6 What are Execution Standards and how do I use them?

These documents have been produced in support of the Eurocodes and are applicable to designs in accordance with the Eurocodes.

The Masonry Eurocode includes its own execution part (BS EN 1996-2) but other areas such as Concrete, Steel, and Geotechnics have separate documents, outside the Eurocodes suite, dealing with execution and workmanship.

7 How will Eurocodes be maintained and developed?

Eurocodes will be maintained and developed by the CEN/TC250 committee. Their responsibilities will include:

- Correction of errors
- Technical and editorial improvements
- Technical amendments with regard to urgent matters of health and safety
- Resolution of questions of interpretation
- Elimination of inconsistencies and misleading statements.

They will also approve any corrigendum (e.g. removal of printing and linguistic errors) or amendment (e.g. modification, addition or deletion of specific parts), as appropriate.

In addition, future editions of the Eurocodes, such as new annexes or parts and eventually new Eurocodes will be needed to include guidance reflecting new European Union policies, innovative design methods, construction techniques, new materials, products and the like.

8 What are the benefits of using the new Eurocodes?

- They will facilitate the acquisition of public sector contracts
- They will facilitate the acquisition of European contracts
- They are among the most advanced technical views prepared by the best informed groups of experts in their fields across Europe
- They are the most comprehensive treatment of subjects, with many aspects not previously codified now being covered by agreed procedures
- They provide a design framework and detailed implementation rules which are valid across Europe and likely to find significant usage worldwide
- They provide common design criteria and methods of meeting necessary requirements for mechanical resistance, stability and resistance to fire
- They provide a common understanding regarding the design of structures between owners, operators and users, designers, contractors and manufacturers of construction products
- They facilitate the marketing and use of structural components and kits in EU Member States
- They facilitate marketing and use of materials and constituent products, the properties of which enter into design calculations

- They enable the preparation of common design aids and software
- They increase competitiveness of European civil engineering firms, contractors, designers and product manufacturers in their global activities.

9 Have all of the Eurocodes been published?

Yes, BSI has now published all of the harmonized codes and national annexes.

The British Standards referred to in Part A of the Building Regulations will be withdrawn on the 31st of March 2010 and be replaced by a new, more technologically sophisticated set of British Standards – the Eurocodes.

10 What happens to the standards I currently use?

Following publication of a European standard, BSI is obliged to withdraw conflicting standards i.e. those within the same scope and field of application as the European standard. Where the national standard is not in a one-to-one relationship with the European standard, the national standard will be amended or revised to delete the conflicting requirements and to reflect the changed scope.

Withdrawn documents are still available and remain in the BSI catalogue for historical information purposes but a BSI committee no longer maintains withdrawn standards. That means that there is no 5-year review when a committee considers the currency of a standard and decides whether to confirm, revise or withdraw it.

11 What happens if I continue to use the old British Standards?

BSI committees have already stopped updating the British Standards to be withdrawn on the 31st of March 2010, so designers need to be mindful of insurance and liability issues if they continue to use them.

The new standards will become the preferred means of demonstrating compliance under the Public Contracts Regulations 2006 and the Construction Products Directive.

12 Is there a legal or insurance-related risk arising from continuing to use the old British Standards?

In any legal proceedings relating to structural design, the courts and other dispute-resolution forums will refer to Eurocodes – the state-of-the-art standards – to reach their decisions. Continuing to use withdrawn standards could put structural designers and their insurers at increasing risk.

There is a risk that with a dual system engineers will use codes to suit themselves and this could introduce further confusion and risk.

13 Which projects use Eurocodes?

The choice of which standards to use will be influenced by EU Directives such as those on public procurement and construction products, which are enacted in the UK as the Public Contracts Regulations 2006 and the Construction Products Regulations 1991 respectively. As such, most UK public sector organizations, utilities and product manufacturers intend to use Eurocodes for all new designs after April 2010.

The Highways Authority (England Wales and Northern Ireland) will expect new designs to be in accordance with Eurocodes after March 2010. The Highways Authority requirements will be described in an Advice Note (an IAN) which will be published shortly. The actual standards to be used on a project will be defined in the AIP (Approval in Principle) document for each contract.

Network Rail will require new work from March 2010 to be in accordance with Eurocodes.

14 Has Eurocode implementation been held up by the delay to the revision of Approved Document A?

A revision to Part A to update the referenced standards has been delayed for unrelated reasons and CLG remains fully supportive of the new British Standards.

There is nothing to stop designers using British Standards cited in the Regulations, it is ‘legally permissible’ to use them, though they should be aware of the comments in Q12.

15 Many engineers are not ready for the new British Standards, why does BSI not postpone withdrawal?

Both BSI and the Government have a legal obligation to meet the agreed date for Europe-wide implementation of the Eurocodes (i.e. 31 March 2010). The CEN agreement to create and apply harmonized standards is made between European governments and then delegated to their National Standards Bodies.

16 How can I purchase Eurocodes?

Eurocodes are published and sold in each country by the National Standards Body and in the United Kingdom can be purchased from BSI at http://shop.bsigroup.com/eurocodes.

17 What kind of guidance on Eurocodes is available from BSI?

Eurocode core documentation

BSI has published all 58 Eurocodes with national annexes, associated NCCI and PD. See the Eurocodes website for more information http://shop.bsigroup.com/eurocodes.

New online managed collection

BSI has recently made available a managed PDF collection of the full set of Eurocodes and national annexes. More information can be requested at http://shop.bsigroup.com/eurocodesmanagedcollection

Commentary, guidance, master classes, conferences

BSI has designed a series of master classes, publications and an annual conference on key Eurocode themes covering key design materials such as concrete, steel, timber.

Further information can be received from http://shop.bsigroup.com/eurocodes

View from the industry – benefits, threats and UK plc's state of readiness

Chris Hendy, Atkins plc

The Eurocodes are widely regarded as the most technically advanced suite of structural design codes available internationally. Why then is it often perceived that progress towards their adoption has been slow in the UK?

There is undoubtedly still some resistance from pockets of the UK structural community. Part of the inertia comes from the fact that the UK has extremely good British Standards already. For example, BS 5400-3 is widely considered to be the most *comprehensive* steel code of practice in the world but few would describe it as the most *economic*. Some in the UK argue that the Eurocode rules go too far and are, in some isolated cases, unsafe. There is however no evidence of this, particularly when the UK national annex has in a few places tightened up requirements. Arguments that the Eurocodes are unsafe because they give different answers to previous British codes are simply unsound and in places the British Standards are far too conservative and are increasingly being shown to be so.

Other resistance stems from the perceived effort involved in the changeover. The Eurocode awareness seminars that have been held over the last few years may potentially have been counter-productive. They have been intended to reassure, whilst at the same time demonstrate there is work to do. In some cases, pointing out a long list of differences in practice has made the process of adoption appear more daunting than perhaps it really is.

While there may be some resistance from within industry, BSI and the Highways Agency are actively driving implementation. The speed of production of national annexes has been on a par with or better than the progress made by much of mainland Europe. In addition, an increasing number of consultants are using Eurocodes to form the basis of departures from standards in the assessment of existing structures because they can improve predicted load carrying resistance.

The state of readiness of industry bodies, software houses and institutions is also excellent by comparison with our other European counterparts. The Concrete Centre and Steel Construction Institute have produced, and continue to produce, much guidance and training material. Many of the big software houses are on top of software upgrades, waiting only for final national annexes to finalize releases. The ICE and IStructE are running seminars and training and publishing a comprehensive set of designers' guides to the various Eurocode parts.

Readiness amongst designers is however more patchy. Some of the big consultants have strategies in hand for helping their engineers to make the transition. Atkins for example has rolled out a series of four-day training courses to 60 'Champions' across the UK and ensured that all other staff have received the same training via a cascade from these Champions. Other companies are planning or have executed similar strategies. However, a significant number of companies are only just starting to consider the issue. There are good reasons to take the change seriously and act quickly. Some of these are discussed below. Most relate to the need to remain competitive.

Steel design

The rules given in the Eurocodes reflect modern research and bring together steel design practices from around Europe. Therefore, for bridges, for example, there is a significant change to the requirements set out in previous UK practice through BS 5400-3. Some typical examples include:

- *Class 4 beams with longitudinal stiffeners* – these are treated in the same way as beams without longitudinal stiffeners in EN 1993, unlike in BS 5400-3 where a completely different approach to calculation was employed involving checking individual panels and stiffeners for buckling in isolation. This allowed little load shedding between components and a single overstressed component could govern the design of the whole cross-section. In EN 1993-1-5, this does not happen and it is the strength of the whole cross-section which is important.
- *Shear–moment interaction* – EN 1993 produces a more economic check of shear and moment interaction than does BS 5400. There are various

reasons for the improvement in economy but the main gain relates to bridge girders for which recent non-linear parametric studies have shown little interaction between shear and bending for Class 3 and 4 cross-sections, and this is reflected in the shape of the interaction curve in EN 1993-1-5

- *Web transverse stiffeners* – The design requirements for web transverse stiffeners, provided to enhance shear resistance, are much less onerous than those of BS 5400 and themselves have still been shown to be conservative [1].

Various pilot studies were conducted for the UK Highways Agency to gauge the difference in resistances overall between BS 5400-3 and EN 1993 and hence measure the differences in expected materials costs. The conclusion was that if the simple application rules were followed, steel bridges with cross-sections in Class 1 and 2 throughout would require very similar quantities of materials for both codes. Where the bridge was more typical, with cross-sections in Class 3 or 4, a typical reduction in materials of around 10% was expected with EN 1993. However, if more advanced analysis techniques are used, such as non-linear analysis, much greater reductions can be achieved.

Concrete design

The rules developed for concrete design also reflect more modern research and reflect the modern use of higher grades of concrete. The formulae given in the Eurocodes use significantly higher concrete strengths than previous UK practice; C70/85 for bridges and C90/105 for buildings. The UK national annex however places a limit on cylinder strength in calculations of 50 MPa for shear due to concerns over the validity of the equations with high strength concrete, particularly those with limestone aggregates.

As with steel design, UK designers can expect to find some differences in resistances between codes. Some typical examples include:

- *Resistance to bending and axial force* – The use of a design reinforcement stress–strain curve allowing for strain hardening in EN 1992 can lead to around 7% greater bending resistance with Class B reinforcement than is obtained with BS 5400-4 where consideration of strain hardening is not permitted. Greater increase is obtained with Class C reinforcement which is more ductile.
- *Shear resistance* – Where there are shear links included in the design, the approach in EN 1992 differs from that in BS 5400-4 and leads to a

potential large increase in economy for reinforced concrete beams. Unlike the BS 5400 truss model employed which has a fixed truss angle of 45°, the truss angle in the Eurocodes can be varied between 45° and 21.8° resulting in up to 2.5 times more resistance provided by the links. This has to be balanced by a potential increase in longitudinal reinforcement where this reinforcement is curtailed, but the designer has far greater choice over where the reinforcement is to be provided and its total quantity.
- *Punching shear* – One significant additional requirement in the Eurocodes involves the calculation of punching shear resistance allowing for the interaction with coexistent bending moment transmitted at the same time as the shear load. A typical example is a pad foundation supporting a column. This is one area where Eurocode 2 produces a lower resistance typically than did BS 5400-4.

Increased use of finite element analysis

The use of finite element (FE) analysis will increase in the UK with the introduction of the Eurocodes as they provide codified rules for the use of both elastic and non-linear analysis which were not previously covered by UK codes; they were not prohibited but approval could be a long process with no guarantee of acceptance. Additionally, the format used in the Eurocodes (particularly steel) often facilitates the use of FE models and, in some situations, using an FE model is the most economic method both in terms of design cost and in terms of material costs.

Designers will need to embrace these analysis methods to remain competitive. FE analysis can give a very accurate representation of the true behaviour of the structure, but only if the assumptions made accurately represent this behaviour. As such, results can be either unsafe or overly safe if the assumptions are incorrect. Some examples of uses that are likely to become common are set out below, together with some discussion on possible pitfalls.

Linear elastic FE analysis is attractive because it permits the principle of superposition to be adopted; influence surfaces can be generated for the effect to be investigated and the results of different loadings may be combined. Elastic finite element modelling is appropriate for calculations on fatigue stress and serviceability where it is desirable for materials to remain elastic, but may be very conservative for predicting ultimate strength where plastic redistribution is possible after first yield. In the Eurocodes, particularly EN 1993, elastic critical buckling analysis will be increasingly used to determine slendernesses for buckling directly from the computer.

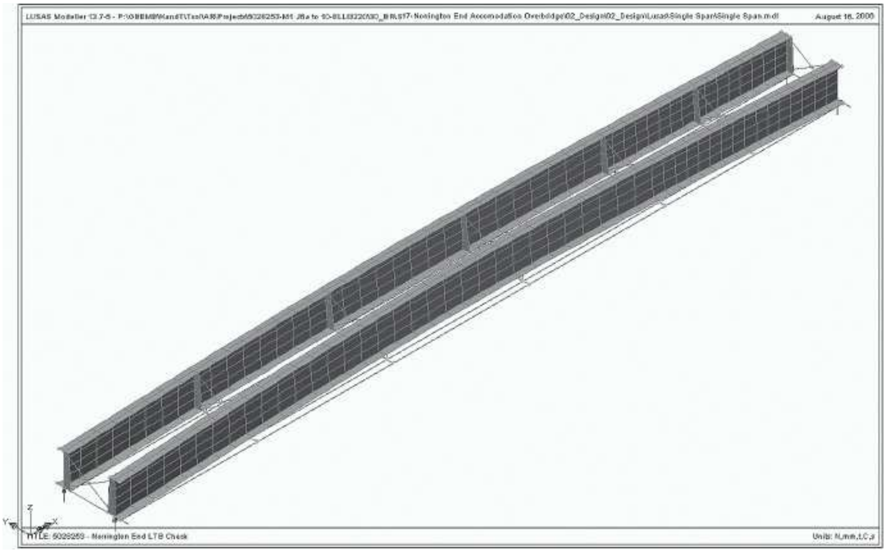
Elastic critical buckling analysis is particularly useful for analysing the construction condition of paired beams before the concrete is poured to make them composite. The slenderness can be determined directly from the elastic critical moment, M_{cr}, according to

$$\bar{\lambda}_{LT} = \sqrt{W_y f_y / M_{cr}}$$

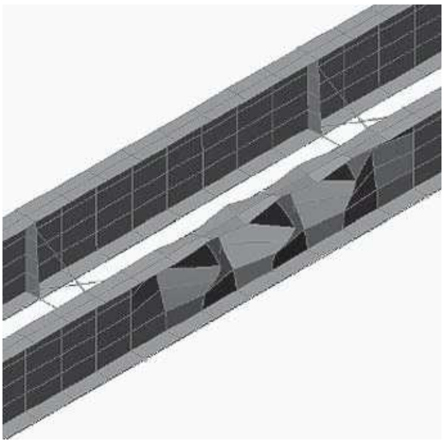
This is quick and easy to do and it is common for the Eurocodes to give significantly greater resistance than BS 5400. Figure 1a shows the critical mode representing global instability of a typical pair of cross-braced beams (the lateral buckling referred to in the code) but this was the twentieth mode produced by the computer; there were numerous lower local buckling modes of the form shown in Figure 1b which could safely be ignored as they were included elsewhere in the codified section properties for the beam. Reference [2] contains an example where a 53% greater ultimate resistance against buckling was produced using this EN 1993 approach rather than BS 5400-3. The analysis of arches also lends itself to the use of elastic critical buckling analysis in a similar manner where determination of the buckling slenderness via an effective length would otherwise be imprecise and necessarily conservative.

Non-linear analysis is the most advanced calculation procedure now permitted by Eurocodes. When performed correctly, non-linear analysis of structures can get very close to the true resistance. This is especially true of steel structures where the ultimate behaviour of steel can be very accurately represented in computer models – *Numerical validation of simplified theories for design rules of transversely stiffened plate girders* [1] covering transversely stiffened plate girders provivdes a good example. The accuracy of reinforced concrete models is less uniform; predominantly flexural behaviour (such as pier second order analysis shown as follows in Figure 2) is well modelled but more complex behaviour requiring prediction of reinforced concrete behaviour under general stress fields is less well understood and predictions show more scatter from test results.

The paired beams above provide an example of the further reserve of strength than can be obtained by using a non-linear model. For the same example, non-linear analysis gave 99% more ultimate resistance than did the simplified approach in BS 5400-3. The reasons for this increased resistance are discussed in *Lateral buckling of plate girders with lateral restraints* [2].



(a) Global buckling mode



(b) Typical local buckling mode

Figure 1. Elastic critical buckling analysis of paired steel beams

Buckling of slender piers by non-linear analysis can also bring significant savings in reinforcement compared to simplified code formulae, such as those in BS 5400-4. A typical example was the piers of the Medway Bridge [3]. The rules for non-linear analysis in EN 1992, including imperfections and material properties, were employed in the design after the initial reinforcement tonnage produced in accordance with BS 5400 was found to be excessive.

The pier shown in Figure 2 was analysed twice; once with 32 mm diameter reinforcement (T32) and again with 40 mm diameter (T40). The deflections shown were for:

- uncracked linear elastic analysis;
- second order uncracked elastic analysis;
- cracked second order analysis with T32 reinforcement;
- cracked second order analysis with T40 reinforcement.

The non-linear analysis resulted in a saving of reinforcement of approximately 60% compared to the UK design code.

There is little guidance available on the use of FE which makes experience in the field very important for successful modelling. The encouragement to use FE modelling by the Eurocodes is likely to lead to more inexperienced designers using it as a routine design tool. Engendering the need for checking strategies in these engineers is therefore extremely important and this can be difficult where the modelled behaviour is complex. Contrary to what many designers believe, the availability of software packages to perform these analyses requires a much greater degree of structural understanding, not a lesser degree, in order to check the model is performing satisfactorily. The example above of elastic critical buckling in paired beams is a case in point; the designer

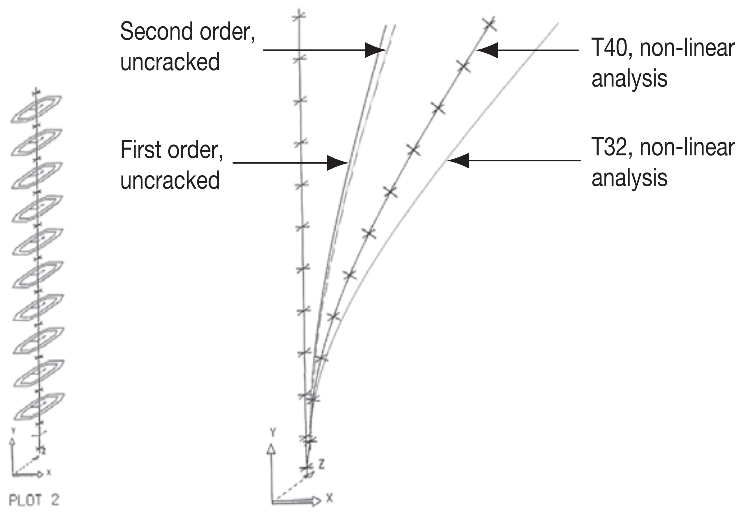


Figure 2. Second order analysis of slender piers

should have a strategy for verifying that the buckling modes produced are realistic and their eigenvalues are the right magnitudes. Standard textbook formulae could, for example, be used to approximate and check the critical stresses for the local buckling modes. A strategy for managing this change needs to be in place in design offices.

The above discussions set out some good reasons to embrace the change quickly. Designers who are not prepared face a risky transition period. The introduction of Eurocodes will provide a common set of design codes for use across Europe and, as considered below, in a number of countries outside Europe. Apart from a unique national annex (which can provide very limited information and will thus be very easy to assimilate by foreign competitors), a design done in the UK will follow the same set of rules as one done elsewhere in Europe. This will facilitate competition by UK designers across a wide range of countries but, of course, the reverse will also be true. If we are slow to adapt in the UK, others will not be and this brings potential threats to our industry.

The threats will not only come from within Europe. Countries with an existing reliance on, or close link to, British Standards are either already committed to adopting Eurocodes (e.g. Malaysia and Singapore) or are weighing up the benefits of adopting them (e.g. Hong Kong). In addition, training is starting in these countries. For example, the Institution of Engineers Malaysia commissioned Atkins to run a two-day Eurocode concrete bridge design training course for 85 delegates in Kuala Lumpur in September 2007, then commissioned another for steel design in March 2008 and has booked subsequent courses. At the time of writing there is no similar-scale external training taking place in the UK in bridge design. These countries may take a keen interest in UK opportunities.

The introduction of Eurocodes and the increased technical sophistication they bring is timely given the growing importance of the sustainability agenda and the drive for leaner construction. Many of the basic application rules in the Eurocodes lead to a modest but significant improvement in economy compared to existing British Standards. In many cases, this is derived from more recent research and testing. However, designers that follow the more complex methods of analysis permitted by the high level principles, such as non-linear analysis, may find very considerable improvements in economy. This will be the case, for example, for slender concrete piers or slender steel panels.

So to return to the original question, we shouldn't consider that the performance of UK plc in adopting Eurocodes has been sluggish. We should however recognize that the Eurocodes bring both opportunities and threats, and so to maximize the former and mitigate the latter now is the time to step up our preparation activities.

References

[1] Presta F., Hendy C.R. *Numerical validation of simplified theories for design rules of transversely stiffened plate girders*, The Structural Engineer, Volume 86, Number 21, pp37–46 (4/11/2008)

[2] Hendy C.R. and Jones R.P. *Lateral buckling of plate girders with lateral restraints,* ICE Bridge Engineering, March 2009

[3] Hendy C.R. and Smith D.A. *Design of the New Medway Bridge, England*, ICE Bridge Engineering, 157, March 2004, Issue BE1, pp27–36

Complete Eurocode listing

The following tables show the constituent parts of the Eurocodes and their corresponding UK national annexes.

This listing is correct at the time of going to print. For the very latest information please go to www.bsigroup.com/eurocodes

NOTE All the Eurocodes listed have separate National Annexes with the following exceptions:

†Eurocodes which include National Annex information in the National Foreword of each Eurocode.

§Eurocodes which do not have National Annexes at all.

Eurocode. BS EN 1990 – Basis of structural design

Eurocode part	Title
BS EN 1990:2002	Basis of structural design
BS EN 1990:2002 Annex A2	Basis of structural design including Amendment A1 for Annex A2 for Bridges

Eurocode 1. BS EN 1991 – Actions on structures

Eurocode part	Title
BS EN 1991-1-1:2002	Actions on structures. General actions. Densities, self-weight, imposed loads for buildings
BS EN 1991-1-2:2002	Actions on structures. General actions. Actions on structures exposed to fire

Eurocode 1. BS EN 1991 – Actions on structures. (*Contd*)

Eurocode part	Title
BS EN 1991-1-3:2003	Actions on structures. General actions. Snow loads
BS EN 1991-1-4:2005	Actions on structures. General actions. Wind actions
BS EN 1991-1-5:2003	Actions on structures. General actions. Thermal actions
BS EN 1991-1-6:2005	Actions on structures. General actions. Actions during execution
BS EN 1991-1-7:2006	Actions on structures. General actions. Accidental actions
BS EN 1991-2:2003	Actions on structures. Traffic loads on bridges
BS EN 1991-3:2006	Actions on structures. Actions induced by cranes and machinery
BS EN 1991-4:2006	Actions on structures. Silos and tanks
PD 6688-1-1	Background paper to the UK National Annex to BS EN 1991-1-1
PD 6688-1-2:2007	Background paper to the UK National Annex to BS EN 1991-1-2
PD 6688-1-4	Background information to the National Annex to BS EN 1991-1-4 and additional guidance
PD 6688-1-5	Background paper to the UK National Annex to BS EN 1991-1-5
PD 6688-1-7	Recommendations for the design of structures to BS EN 1991-1-7
PD 6688-2	Recommendations for the design of structures to BS EN 1992-2

Eurocode 2. BS EN 1992 – Design of concrete structures

Eurocode part	Title
BS EN 1992-1-1:2004	Design of concrete structures. General rules and rules for buildings

BS EN 1992-1-2:2004	Design of concrete structures. Fire design
BS EN 1992-2:2005	Design of concrete structures. Concrete bridges. Design and detailing rules
BS EN 1992-3:2006	Design of concrete structures. Liquid retaining and containing structures
PD 6687:2006	Background paper to the UK National Annexes to BS EN 1992-1
PD 6687-1	Background paper to the UK National Annexes to BS EN 1992-1 and BS EN 1992-3 (supersedes PD 6687:2006)
PD 6687-2:2007	Recommendations for the design of structures to BS EN 1992-2

Eurocode 3. BS EN 1993 – Design of steel structures

Eurocode part	**Title**
BS EN 1993-1-1:2005	Design of steel structures. General rules and rules for buildings
BS EN 1993-1-2:2005	Design of steel structures. General rules. Structural fire design
BS EN 1993-1-3:2006	Design of steel structures. General rules. Supplementary rules for cold-formed members and sheeting
BS EN 1993-1-4:2006	Design of steel structures. General rules. Supplementary rules for stainless steels
BS EN 1993-1-5:2006	Design of steel structures. Plated structural elements
†BS EN 1993-1-6:2007	Design of steel structures. General. Strength and stability of shell structures
†BS EN 1993-1-7:2007	Design of steel structures. General. Plated structures subject to out of plane loading
BS EN 1993-1-8:2005	Design of steel structures. Design of joints
BS EN 1993-1-9:2005	Design of steel structures. Fatigue strength

Eurocode 3. BS EN 1993 – Design of steel structures. (*Contd*)

Eurocode part	Title
BS EN 1993-1-10:2005	Design of steel structures. Material toughness and through-thickness properties
BS EN 1993-1-11:2006	Design of steel structures. Design of structures with tension components
BS EN 1993-1-12:2007	Design of steel structures. Additional rules for the extension of EN 1993 up to steel grades S 700
BS EN 1993-2:2006	Design of steel structures. Steel bridges
BS EN 1993-3-1:2007	Design of steel structures. Towers, masts and chimneys. Towers and masts
§BS EN 1993-3-2:2008	Design of steel structures. Towers, masts and chimneys. Chimneys
†BS EN 1993-4-1:2007	Design of steel structures. Silos, tanks and pipelines. Silos
†BS EN 1993-4-2:2007	Design of steel structures. Silos, tanks and pipelines. Tanks
†BS EN 1993-4-3:2007	Design of steel structures. Silos, tanks and pipelines. Pipelines
BS EN 1993-5:2007	Design of steel structures. Piling
BS EN 1993-6:2007	Design of steel structures. Crane supporting structures
PD 6695-1-9	Recommendations for the design of structures to BS EN 1993-1-9
PD 6695-1-10	Recommendations for the design of structures to BS EN 1993-1-10
PD 6695-2	Recommendations for the design of bridges to BS EN 1993

Eurocode 4. BS EN 1994 – Design of composite steel and concrete structures

Eurocode part	Title
BS EN 1994-1-1:2004	Design of composite steel and concrete structures. General rules and rules for buildings
BS EN 1994-1-2:2005	Design of composite steel and concrete structures. General rules. Structural fire design
BS EN 1994-2:2005	Design of composite steel and concrete structures. General rules and rules for bridges
PD 6696-2:2007	Recommendations for the design of structures to BS EN 1994-2:2005

Eurocode 5. BS EN 1995 – Design of timber structures

Eurocode part	Title
BS EN 1995-1-1:2004	Design of timber structures. General. Common rules and rules for buildings
BS EN 1995-1-2:2004	Design of timber structures. General. Structural fire design
BS EN 1995-2:2004	Design of timber structures. Bridges

Eurocode 6. BS EN 1996 – Design of masonry structures

Eurocode part	Title
BS EN 1996-1-1:2005	Design of masonry structures. General rules for reinforced and unreinforced masonry structures
BS EN 1996-1-2:2005	Design of masonry structures. General rules. Structural fire design.
BS EN 1996-2:2006	Design of masonry structures. Design considerations, selection of materials and execution of masonry
BS EN 1996-3:2006	Design of masonry structures. Simplified calculation methods for unreinforced masonry structures

Eurocode 7. BS EN 1997 – Geotechnical design

Eurocode part	Title
BS EN 1997-1:2004	Geotechnical design. General rules
BS EN 1997-2:2007	Geotechnical design. Ground investigation and testing
PD 6694-1:2007	Recommendations for the design of structures subject to traffic loading to BS EN 1997-1:2004

Eurocode 8. BS EN 1998 – Design of structures for earthquake resistance

Eurocode part	Title
BS EN 1998-1:2004	Design of structures for earthquake resistance. General rules, seismic actions and rules for buildings
BS EN 1998-2:2005	Design of structures for earthquake resistance. Bridges
§BS EN 1998-3:2005	Design of structures for earthquake resistance. Assessment and retrofitting of buildings
BS EN 1998-4:2006	Design of structures for earthquake resistance. Silos, tanks and pipelines
BS EN 1998-5:2004	Design of structures for earthquake resistance. Foundations, retaining structure and geotechnical aspects
BS EN 1998-6:2005	Design of structures for earthquake resistance. Towers, masts and chimneys
PD 6698:2009	Background paper to the UK National Annexes to BS EN 1998-1, BS EN 1998-2, BS EN 1998-4, BS EN 1998-5 and BS EN 1998-6

Eurocode 9. BS EN 1999 – Design of aluminium structures

Eurocode part	Title
BS EN 1999-1-1:2007	Design of aluminium structures. General structural rules
BS EN 1999-1-2:2007	Design of aluminium structures. Structural fire design
BS EN 1999-1-3:2007	Design of aluminium structures. Structures susceptible to fatigue
BS EN 1999-1-4:2007	Design of aluminium structures. Cold-formed structural sheeting
BS EN 1999-1-5:2007	Design of aluminium structures. Shell structures
PD 6702-1:2009	Structural use of aluminium. Recommendations for the design of aluminium structures to BS EN 1999
PD 6705-3:2009	Structural use of steel and aluminium. Recommendations for the execution of aluminium structures to BS EN 1090-3

Key aspects of the Eurocodes

- The Eurocodes support National Building Regulations and other national requirements for regulated work but remain subservient to them.
- National regulations set the appropriate level of safety through Nationally Determined Parameters (NDP). Certain other parameters can be set by individual countries.
- The clauses in the Eurocodes are divided into principles and application rules. Principles are identified by (P) after the clause number and cover items for which no alternative is permitted. Application rules are recommended methods of achieving the principles but alternative rules may also be used.
- There are two types of annex in the Eurocodes. Normative annexes are part of the requirements of the code.
- Informative annexes provide guidance that can be adopted or not on a country by country basis.
- The national annex is a special type of informative annex that contains the choices made by a particular country. Typically the national annex will state values and classes applicable to that country, provide value where only a symbol is given in the Eurocode and provide country specific data. The national annex also chooses when alternatives are given in the Eurocodes and indicates which informative annexes may be used. Finally it refers to Non-Contradictory Complementary Information (NCCI).
- An NCCI is a way of introducing additional guidance to supplement the Eurocodes without contradicting them.

Eurocode: Basis of structural design

Professor Haig Gulvanessian CBE, Civil Engineering and Eurocode Consultant

Introduction

This chapter gives a brief introduction to EN 1990, describes its main innovative features, particularly the requirements, reliability differentiation, based on the consequences of failure, limit state and actions and the choice of load combination format and safety factors by the UK national annex considering consistency of safety for all materials.

EN 1990 (*Eurocode 0: Basis of structural design*) is the head key code for the harmonized Structural Eurocodes. EN 1990 establishes and provides comprehensive information and guidance for all the Structural Eurocodes, the principles and requirements for safety and serviceability, and provides the basis and general principles for the structural design and verification of buildings and civil engineering structures (including bridges, towers and masts, silos and tanks etc.). EN 1990 gives guidelines for related aspects of structural reliability, durability and quality control. It is based on the limit state concept and used in conjunction with the partial factor method. Comprehensive background information is given on EN 1990 by Gulvanessian, Calgaro and Holicky [1].

As shown in Figure 0.1, EN 1990 will be used with every Eurocode part for the design of new structures, together with:

- EN 1991 (Eurocode 1: *Actions on structures*); and
- EN 1992 to EN 1999 (design Eurocodes 2 to 9)

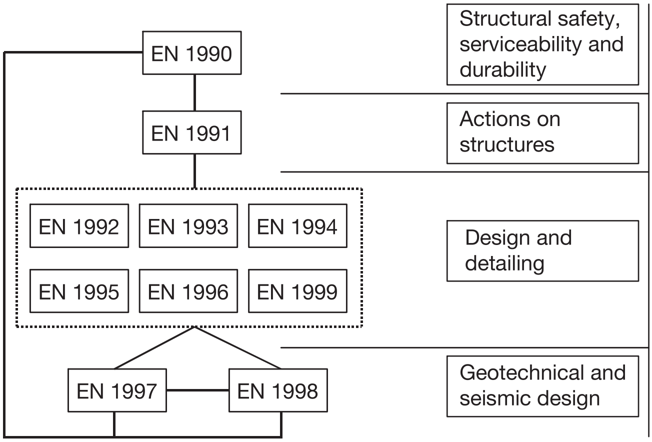


Figure 0.1. Links between Eurocodes

This is different to the situation adopted by the present British Standard codes of practice (e.g. BS 8110, BS 5950, BS 5628 etc.) because with the design Eurocodes the requirements for achieving safety, serviceability and durability and the expressions for action effects for the verification of ultimate and serviceability limit states and their associated factors of safety are only given in EN 1990. Unlike the equivalent British Standard codes of practice the material Eurocodes (EN 1992, EN 1993, EN 1994, EN 1995, EN 1996 and EN 1999) only include clauses for design and detailing in the appropriate material and require all the material independent information for the design (e.g. safety factors for actions, load combination expressions etc.) from EN 1990.

Furthermore, construction products requiring CE marking (e.g. pre-cast concrete products, metal frame domestic houses, timber frame housing etc.) all need to use the principles and rules in EN 1990 together with the appropriate Eurocodes, thus ensuring a level playing field, as do the execution standards.

As well as being the key Eurocode in setting recommended safety levels, EN 1990 also introduces innovative aspects described as follows which encourage the design engineer to consider the safety of people in the built environment together with the responsible consideration of economy by:

- allowing reliability differentiation based on the consequences of failure;
- introducing the concept of using the representative values of actions and not only the characteristic values as used for UK codes of practice. The

loads used in the EN 1990 load combinations recognize the appropriate cases where:
- rare
- frequent, or
- quasi-permanent

occurring events are being considered with the use of an appropriate reduction coefficient (ψ), applied to the characteristic load values as appropriate. The use of the representative values for actions in the load combination expressions for ultimate and serviceability limit state verifications are logical and give economies for particular design situations;

- an alternative load combination format, giving the choice to the designer of using either the expressions 6.10 or 6.10a/6.10b for the combination of actions for ultimate limit state verification. This choice provides opportunities for economy especially for the heavier materials, and can provide flexibility with regard to assessment;
- the use of lower factors of safety for loads compared to British Standards. Although the effects of actions according to the Eurocodes are lower than British Standards codes for ULS and SLS verification, this should not be a concern to the industry as the EN 1990 values are based on better science and better research;
- the use of advanced analytical techniques for the designer is encouraged and the use of probabilistic methods should the designer wish to use these for the more specialist design problems.

Principal objectives of EN 1990

EN 1990's principal objective is that this Eurocode sets out for every Eurocode part the *principles* and *requirements* for achieving:

- safety;
- serviceability;
- durability

of structures.

EN 1990 provides the information for safety factors for actions and the combination for action effects for the verification of both ultimate and serviceability limit states. Its rules are applicable for the design of building and civil engineering structures including bridges, masts, towers, silos, tanks, chimneys and geotechnical structures.

The requirements of EN 1990

To achieve safety, serviceability and durability for structures EN 1990 contains requirements to be adhered to by the complete Eurocode suite and construction product standards on

- fundamental requirements (safety, serviceability, resistance to fire and robustness);
- reliability management and differentiation;
- design working life;
- durability;
- quality assurance and quality control.

Each requirement is described as follows.

Fundamental requirements

The fundamental requirements stipulate that:

a) a structure should be designed and executed in such a way that it will, during its intended life with appropriate degrees of reliability and in an economic way:
   - sustain all actions and influences likely to occur during execution and use (safety requirement); and
   - meet the specified serviceability requirements for a structure or a structural element (serviceability requirement); and

b) in the case of fire, the structural resistance should be adequate for the required period of time;

c) a structure should be designed and executed in such a way that it will not be damaged by events such as explosion, impact or consequences of human errors, to an extent disproportionate to the original cause (robustness requirement). EN 1990 provides methods of avoiding or limiting potential damage.

Reliability differentiation

Design and execution according to the suite of the Eurocodes, together with appropriate quality control measures, will ensure an appropriate degree of reliability for the majority of structures. EN 1990 provides guidance for adopting a different level of reliability (reliability differentiation). Additional guidance is included in an informative annex to EN 1990 *Management of structural reliability for construction works*. Calgaro and Gulvanessian [2] describe the management of structural reliability in EN 1990, where the concept of the risk background of the Eurocodes is described more comprehensively.

EN 1990 gives recommended values of partial factors applicable to actions and provides a framework for the management, at national levels, of structural reliability. Embodied in the values of the partial factors are implicit 'acceptable' or 'accepted' risk levels, which relates to the consequences of the hazard and use of the structure. Risk may be defined as:

$\text{Prob}(F) \times C$

where:

Prob(F) is the probability of the hazard occurring, and
C is the consequence in magnitude or extent, expressed, for example, in numbers of deaths, time or monetary units.

Partial factor design is based on the consideration of limit states which are, in most common cases, classified into ultimate and serviceability limit states idealizing undesirable phenomena. The design is such that their probability of occurrence in 50 years is less than an 'acceptable' value.

Figure 0.2 shows the ranges of values for probabilities for the ultimate and serviceability limit states, obtained when using EN 1990.

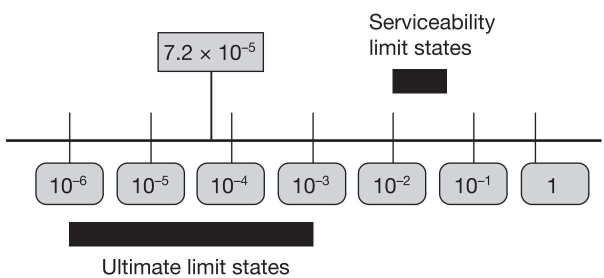


Figure 0.2. Probabilities associated with limit states

Table 0.1. Design working life classification

Design working life category	Indicative design working life (years)	Examples
1	10	Temporary structures
2	10–25	Replaceable structural parts, e.g. gantry girders, bearings
3	15–25	Agricultural and similar buildings
4	50	Buildings and other common structures
5	120	Monumental buildings, bridges and other civil engineering structures

Design working life

In EN 1990 the design working life is the assumed period for which a structure is to be used for its intended purpose with anticipated maintenance but without major repair being necessary. Table 0.1 taken from the UK national annex to EN 1990 gives a design working life classification.

Durability

EN 1990 stipulates that the structure should be designed such that deterioration over its design working life does not impair the performance of the structure.

The durability of a structure is its ability to remain fit for use during the design working life given appropriate maintenance. The structure should be designed in such a way, and/or provided with protection so that no significant deterioration is likely to occur within the period between successive inspections. The need for critical parts of the structure to be available for inspection without complicated dismantling should be considered in the design. Other interrelated factors that should be considered to ensure an adequately durable structure are listed in EN 1990 and each is considered and given as follows:

a) the intended and future use of the structure;
b) the required performance criteria;

c) the expected environmental influences;
d) the composition, properties and performance of materials;
e) the choice of a structural system;
f) the shape of members and structural detailing, and buildability;
g) the quality of workmanship and level of control;
h) the particular protective measures;
i) the maintenance during the intended life.

Quality assurance and quality control

EN 1990 stipulates that appropriate quality assurance measures should be taken in order to provide a structure that corresponds to the requirements and to the assumptions made in the design by:

- definition of the reliability requirements;
- organizational measures;
- controls at the stages of design, execution, use and maintenance.

Principles of limit state design

Ultimate and serviceability limit states

EN 1990 is based on the limit state concept used in conjunction with the partial safety factor method where limit states are the states beyond which the structure no longer fulfils the relevant design criteria. Two different types of limit state are considered, namely ultimate limit state and serviceability limit state.

Based on the use of structural and load models, it has to be verified that no limit state is exceeded when relevant design values for actions, material and product properties, and geometrical data are used. This is achieved by the partial factor method.

In the partial factor method the basic variables (i.e. actions, resistances and geometrical properties) are given design values through the use of partial factors, ψ, and reduction coefficients, γ, of the characteristic values of variable actions (see Figure 0.3).

Design may also be based on a combination of tests and calculations, provided that the required level of reliability is achieved. Alternatively, EN 1990 allows for design directly based on probabilistic methods (see Figure 0.4).

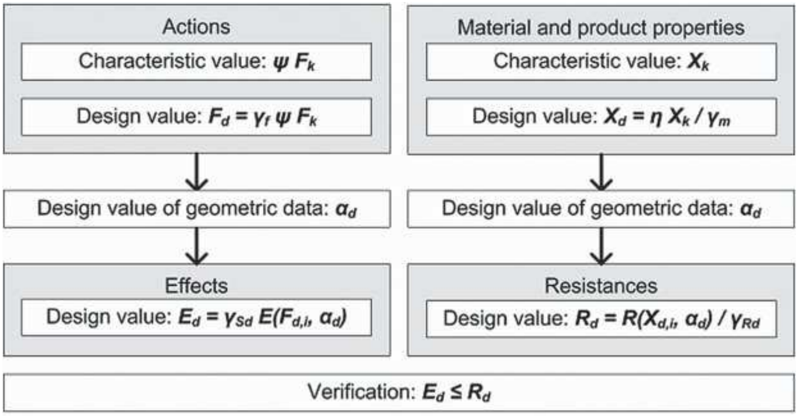


Figure 0.3. Verification by the partial factor method

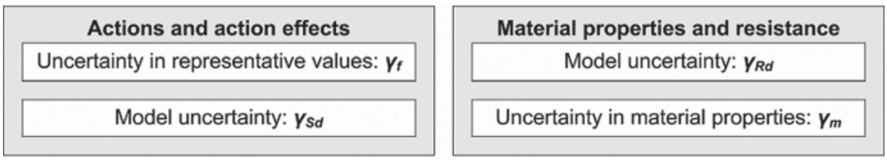


Figure 0.4. Individual partial safety factors

Design situations

Design situations are sets of physical conditions representing the real conditions occurring during the execution and use of the structure, for which the design will demonstrate that relevant limit states are not exceeded.

EN 1990 stipulates that a relevant design situation needs to be selected to take account of the circumstances in which the structure may be required to fulfil its function.

Ultimate limit state verification design situations

EN 1990 classifies design situations for ultimate limit state verification as follows:

- persistent situations (conditions of normal use);
- transient situations (temporary conditions, e.g. during execution);

- accidental situations; and
- seismic situations.

Serviceability limit state verification design situations

Serviceability limit states correspond to conditions beyond which specified service requirements for a structure or structural element are no longer met and the design situations concern:

- the functioning of the construction works or parts of them;
- the comfort of people; and
- the appearance.

EN 1990 recommends that the serviceability requirements should be determined in contracts and/or in the design. EN 1990 distinguishes between reversible and irreversible serviceability limit states. EN 1990 gives three expressions for serviceability design: characteristic, frequent and quasi-permanent.

Actions

Actions are sets of forces, imposed displacements or accelerations. They are classified by their variation in time as follows:

- permanent actions, G, e.g. self-weight of structures, fixed equipment and road surfacing, and indirect actions caused by shrinkage and uneven settlements;
- variable actions, Q, e.g. imposed loads on building floors, beams and roofs, wind actions or snow loads;
- accidental actions, A, e.g. explosions or impact from vehicles.

A variable action has four representative values. In decreasing order of magnitude, they are:

- characteristic value Q_k;
- combination value $\psi_0 Q_k$;
- frequent value $\psi_1 Q_k$;
- quasi-permanent value $\psi_2 Q_k$.

The reduction coefficients, ψ, are applied to the characteristic load values which are appropriate to cases where

- combination (or Rare)
- frequent, or
- quasi-permanent

occurring events are being considered.

Verification by the partial factor method

Ultimate limit states

For the ultimate limit state verification, EN 1990 stipulates that the effects of design actions do not exceed the design resistance of the structure at the ultimate limit state; and the following ultimate limit states need to be verified.

Ultimate limit states concern the safety of people and/or the safety of structures and, in special circumstances, the protection of the contents. They are associated with collapse or with other similar forms of structural failure.

In EN 1990 the following ultimate limit states are verified, where relevant.

- **EQU**. Loss of static equilibrium of the structure or any part of it considered as a rigid body, where:
  - minor variations in the value or the spatial distribution of actions from a single source are significant;
  - the strengths of construction materials or ground are generally not governing.
- **STR**. Internal failure or excessive deformation of the structure or structural members, including footings, piles and basement walls, etc., where the strength of construction materials of the structure governs.
- **GEO**. Failure or excessive deformation of the ground where the strengths of soil or rock are significant in providing resistance.
- **FAT**. Fatigue failure of the structure or structural members. The combinations apply:
  - persistent or transient design situation (fundamental combination);
  - accidental design situation;
  - seismic design situation.

For a limit state of static equilibrium (EQU), it is verified that:

$E_{d,dst} \leq E_{d,stb}$

where

$E_{d,dst}$ is the design value of the effect of destabilizing actions, and
$E_{d,stb}$ is the design value of the effect of stabilizing actions.

When considering a limit state of rupture or excessive deformation of a section, member or connection (STR and/or GEO), it is verified that:

$$E_d \leq R_d$$

where

E_d is the design value of the effect of actions, and
R_d is the design value of the corresponding resistance.

Specific rules for FAT limit states are given in the design Eurocodes EN 1992 to EN 1999.

For the ultimate limit state verification, EN 1990 stipulates that the effects of design actions do not exceed the design resistance of the structure at the ultimate limit state; and the following ultimate limit states need to be verified.

Alternative load combination expressions in EN 1990 for the persistent and transient design situations

EN 1990 specifies three sets of alternative combination expressions for the determination of action effects, expressions 6.10, 6.10a and 6.10b, and 6.10a modified and 6.10b (see the following) for the persistent and transient design situations to be used by EN 1991 and the design Eurocodes for ultimate limit state verification, as follows:

a) $$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} \text{ “+” } \gamma_P P \text{ “+” } \gamma_{Q,1} Q_{k,1} \text{ “+” } \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \qquad (6.10)$$

The procedure using expression 6.10 is denoted as case A in this paper.

For this case the combination of actions is governed by a leading variable action $Q_{k,1}$ represented by its characteristic value and multiplied by its appropriate safety factor γ_Q. Other variable actions $Q_{k,i}$ for $i > 1$ which may act simultaneously with the leading variable action $Q_{k,1}$ are taken into account as accompanying variable actions and are represented by their combination value, i.e. their characteristic value reduced by the relevant combination

$$\overbrace{E_d}^{\text{Design effect}} = \underbrace{E}_{\text{Effect of}} \left[\overbrace{\sum_{j\geq 1}\gamma_{G,j}G_{k,j}}^{\text{Permanent actions}} \underbrace{\text{“+”}}_{\text{Combined with}} \overbrace{\gamma_P P}^{\text{Prestress}} \text{“+”} \overbrace{\gamma_{Q,1}Q_{k,1}}^{\text{Leading variable action}} \text{“+”} \overbrace{\sum_{i\geq 1}\gamma_{Q,i}\psi_{0,i}Q_{k,i}}^{\text{Accompanying variable actions}} \right]$$

Figure 0.5. ULS verification (persistent and transient design situation) expression 6.10

factor, ψ_0, and are multiplied by the appropriate safety factor to obtain the design values.

The permanent actions are taken into account with their characteristic values, and are multiplied by the load factor γ_G. Depending on whether the permanent actions act favourably or unfavourably they have different design values.

This is explained in Figure 0.5.

b) or the less favourable of the two following expressions:

$$\sum_{j\geq 1}\gamma_{G,j}\, G_{k,j} \text{ “+” } \gamma_P P \text{ “+” } \gamma_{Q,1}\,\psi_{0,1}\, Q_{k,1} \text{ “+” } \sum_{i>1}\gamma_{Q,i}\psi_{0,i}\, Q_{k,i} \qquad (6.10a)$$

$$\sum_{j\geq 1}\xi_j\gamma_{G,j}\, G_{k,j} \text{ “+” } \gamma_P P \text{ “+” } \gamma_{Q,1}\, Q_{k,1} \text{ “+” } \sum_{i>1}\gamma_{Q,i}\psi_{0,i}\, Q_{k,i} \qquad (6.10b)$$

The procedure using expressions 6.10a and 6.10b is denoted as case B.

In expression 6.10a, there is no leading variable action: all the variable actions are taken into account with their combination value, i.e. their value is reduced by the relevant combination factor ψ_0. The permanent actions are taken into account as in expression 6.10, and the unfavourable permanent actions may be considered as the leading action in the combination of actions. All the actions are multiplied by the appropriate safety factors, γ_G or γ_Q.

In expression 6.10b the combination of actions is governed by a leading variable action represented by its characteristic value as in expression 6.10 with the other variable actions being taken into account as accompanying variable actions and are represented by their combination value, i.e. their characteristic value is reduced by the appropriate combination coefficient of a variable action ψ_0. But the unfavourable permanent actions are taken into account with a characteristic value reduced by a reduction factor, ξ, which may be considered as a combination factor.

All the actions are multiplied by the appropriate load factors, γ_G or γ_Q. When the envelope of the two expressions showing the less favourable effects of expressions 6.10a and 6.10b is determined, generally expression 6.10a applies to members where the ratio of variable action to total action is low, i.e. for heavier structural materials, and 6.10b applies where the same ratio is high, i.e. for lighter structural materials.

c) or expression 6.10a modified to include self-weight only and expression 6.10b, as shown as follows:

$$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} \text{ “+” } \gamma_P P \tag{6.10a, modified}$$

$$\sum_{j\geq 1} \xi_j \gamma_{G,j} G_{k,j} \text{ “+” } \gamma_P P \text{ “+” } \gamma_{Q,1} Q_{k,1} \text{ “+” } \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \tag{6.10b}$$

The procedure using expressions 6.10a modified and 6.10b is denoted as case C. This case is very similar to case B but the first of pair expressions includes only permanent actions.

In the preceding text:

“+” implies ‘to be combined with’
Σ implies ‘the combined effect of’
ξ is a reduction factor for unfavorable permanent actions G

EN 1990 allows through NDPs and the national annexes for

- the choice of which of the three combination expressions given in EN 1990 to use; and
- the specification of appropriate safety factors, γ, and combination coefficients, ψ and ξ, for actions

which should be used nationally.

Comparison of expressions for the combination of the effects of actions between BSI structural codes of practice and EN 1990 (note that the factors of safety for permanent and variable actions are lower in EN 1990) using expression 6.10, are shown as follows.

For one variable action (imposed or wind)

- BSI: $1.4G_k + (1.4 \text{ or } 1.6)Q_k$
- EN 1990: $1.35G_k + 1.5Q_k$

For two or more variable actions (imposed + wind)

- BSI: $1.2G_k + 1.2Q_{kl} + 1.2Q_{k2}$
- EN 1990: $1.35G_k + 1.5Q_{kl} + 0.75Q_{k2}$

where

G_k is the characteristic value for permanent actions, and
Q_k is the characteristic value for variable actions

Choice of load combination expressions for the UK national annex to EN 1990

Based on an investigation by Gulvanessian and Holický [3], the following two combinations have been adopted in the UK national annex for EN 1990 for buildings.

- Expression 6.10 with $\gamma_G = 1.35$ and $\gamma_Q = 1.5$
- Expressions 6.10a and 6.10b with $\gamma_G = 1.35$, $\gamma_Q = 1.5$ and $\xi = 0.925$

All recommended γ and ψ (except ψ_0 for wind actions, where in the UK national annex $\psi_0 = 0.5$) values have been adopted by the UK national annex for EN 1990, and are generally being adopted by most CEN Member States.

For bridges only the use of expression 6.10 is permitted.

Load combination expressions in EN 1990 for accidental design situations

For the ultimate limit states verification for accidental design situations, EN 1990 requires the following combination expression to be investigated:

$$\sum_{j\geq 1} G_{k,j} \text{ “+” } P \text{ “+” } A_d \text{ “+” } (\psi_{1,1} \text{ or } \psi_{2,1})Q_{k,1} \text{ “+” } \sum_{i>1} \psi_{2,i}Q_{k,i}$$

The choice between $\psi_{1,1}Q_{k,1}$ or $\psi_{2,1}Q_{k,1}$ should be related to the relevant accidental design situation (impact, fire or survival after an accidental event or situation). In the UK national annex to EN 1990, $\psi_{1,1}Q_{k,1}$ is chosen.

The combinations of actions for accidental design situations should either

- involve an explicit accidental action A (fire or impact); or
- refer to a situation after an accidental event ($A = 0$).

For fire situations, apart from the temperature effect on the material properties, A_d should represent the design value of the indirect thermal action due to fire.

The expression for the accidental design situation specifies factors of safety of unity both for the self-weight and the accidental action A_d and a frequent or quasi-permanent value for the leading variable action. The philosophy behind this is the recognition that an accident on a building or construction works is a very rare event (although when it does occur the consequences may be severe) and hence EN 1990 provides an economic solution.

Serviceability limit states

For the serviceability limit states verification, EN 1990 stipulates that:

$$E_d \leq C_d \tag{6.13}$$

where

C_d is the limiting design value of the relevant serviceability criterion, and
E_d is the design value of the effects of actions specified in the serviceability criterion, determined on the basis of the relevant combination.

Combination of actions for the serviceability limit states

For the serviceability limit states verification, EN 1990 requires the three following combinations to be investigated:

a) The characteristic (rare) combination used mainly in those cases when exceedance of a limit state causes a permanent local damage or permanent unacceptable deformation.

$$\sum_{j\geq 1} G_{k,j} \text{ “+” } P_k \text{ “+” } Q_{k,1} \text{ “+” } \sum_{i>1} \psi_{0,i} Q_{k,i} \tag{6.14b}$$

b) The frequent combinations used mainly in those cases when exceedance of a limit state causes local damage, large deformations or vibrations which are temporary.

$$\sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } \psi_{1,1}Q_{k,1} \text{ "+" } \sum_{i>1} \psi_{2,i}Q_{k,i} \qquad (6.15b)$$

c) The quasi-permanent combinations used mainly when long term effects are of importance.

$$\sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } \sum_{i\geq 1} \psi_{2,i}Q_{k,i} \qquad (6.16b)$$

Partial factors for serviceability limit states

Unless otherwise stated (e.g. in EN 1991 to 1999), the partial factors for serviceability limit states are equal to 1.0. ψ factors are given in Table 1.3 in the chapter on Eurocode 1 in this book.

Conclusions

EN 1990 is a fully operative code and the concept of a fully operative material-independent code is new to the European design engineer. It is certainly not a code that should be read once and then placed on the bookshelf. It is the key Eurocode that sets the requirements for design, material, product and execution standards. EN 1990 needs to be fully understood as it is key to designing structures that have an acceptable level of safety and economy, with opportunities for innovation.

A course and a designers guide for EN 1990 are available in the UK through Thomas Telford Ltd. of the Institution of Civil Engineers.

Regarding implementation of EN 1990 in the UK, EN 1990 was published in April 2002 and the UK national annex for buildings was published in 2004. The UK national annex for bridges is due in 2009.

References

[1] Gulvanessian H., Calgaro J.A, Holický M. *Designers' Guide to EN 1990: Eurocode: Basis of Structural Design*, Thomas Telford, London, 2002

[2] Calgaro J.A., Gulvanessian H. Management of reliability and risk in the Eurocode system. Conference – Safety, Risk and Reliability, Malta – Trends in Engineering. IABSE 2001.

[3] Gulvanessian H., Holický M. *Eurocodes: Using reliability analysis to combine action effects. Proceedings of the Institution of Civil Engineers, Structures and Buildings*, Thomas Telford, London, 2005

Eurocode 1: Actions on structures

Professor Haig Gulvanessian CBE, Civil Engineering and Eurocode Consultant

Background

This chapter gives a brief summary of the scope, field of application and difference within UK practice for each Part of EN 1991.

EN 1991 (*Eurocode 1: Actions on structures*) provides comprehensive information and guidance on all actions that it is normally necessary to consider in the design of buildings and civil engineering works. The European Union initiated the preparation of EN 1991 in 1985. All Parts have been published as have all the corresponding UK national annexes. This chapter describes the scope of Eurocode 1 and its main provisions.

EN 1991 comprises 10 Parts as shown in Table 1.1. The background to these ten Parts is comprehensively described by Gulvanessian, Formichi and Calgaro [1] and Calgaro, Tchumi and Gulvanessian [2]. The Parts are referred to in this chapter by their designated EN numbers. These Parts will provide the actions for use with EN 1990 *Eurocode: Basis of structural design*, and EN 1992–EN 1999 as appropriate, for design and verification on the basis of overall principles given in EN 1990.

Difference between EN 1991 and the UK system of loading codes

Each Part of EN 1991 gives unique guidance on a particular type of action. Within each Part, guidance is provided for buildings and other construction works, e.g. bridges. This is different to the British Standard system of loading codes where the codes are based on the type of structure, e.g. BS 6399 for buildings and BS 5400 for bridges.

Table 1.1. The Parts of *Eurocode 1: Actions on structures*

EN Part number	Title of Part
1991-1-1	*Densities, self-weight and imposed loads*
1991-1-2	*Actions on structures exposed to fire*
1991-1-3	*Snow loads*
1991-1-4	*Wind actions*
1991-1-5	*Thermal actions*
1991-1-6	*Actions during execution*
1991-1-7	*Accidental actions due to impact and explosions*
1991-2	*Traffic loads on bridges*
1991-3	*Actions induced by cranes and machinery*
1991-4	*Actions in silos and tanks*

Definition of actions in EN 1990

In EN 1990 and EN 1991, actions are classified by their:

a) variation in time – permanent, variable or accidental;
b) origin – direct or indirect;
c) spatial variation – fixed or free; and
d) nature and/or structural response – static or dynamic.

Actions are described by a model, its magnitude being represented in the most common cases by one scalar, e.g. vehicle axle spacing, and its magnitude is commonly represented by a single scalar. The scalar may adopt several representative values, e.g. a dominant (leading) or non-dominant (accompanying) action. Several scalars are used for multi-component action. More complex representations are required for fatigue and dynamic actions.

Table 1.2 gives examples of the classification of actions with regard to variation in time.

The term single action is also used to define an action which is statistically independent in time and space from any other action acting on the structure.

The self-weight of a structure can be represented by a single characteristic value, (G_k), provided the variability of G is small, and it can be calculated on the basis of the nominal dimensions and the mean unit mass. If the variability of G is not small and the statistical distribution is known, two values are used: an upper value ($G_{k,sup}$) and a lower value ($G_{k,inf}$). More information on this subject has been given by Ostlund [3].

Table 1.2. Classification of actions

Permanent actions	Variable actions	Accidental actions
a) Self-weight of structures, fittings and fixed equipment b) Prestressing force c) Water and soil pressures d) Indirect action, e.g. settlement of supports	a) Imposed floor loads b) Snow loads c) Wind loads d) Indirect action, e.g. temperature effects e) Actions due to traffic	a) Explosions b) Fire c) Impact from vehicles

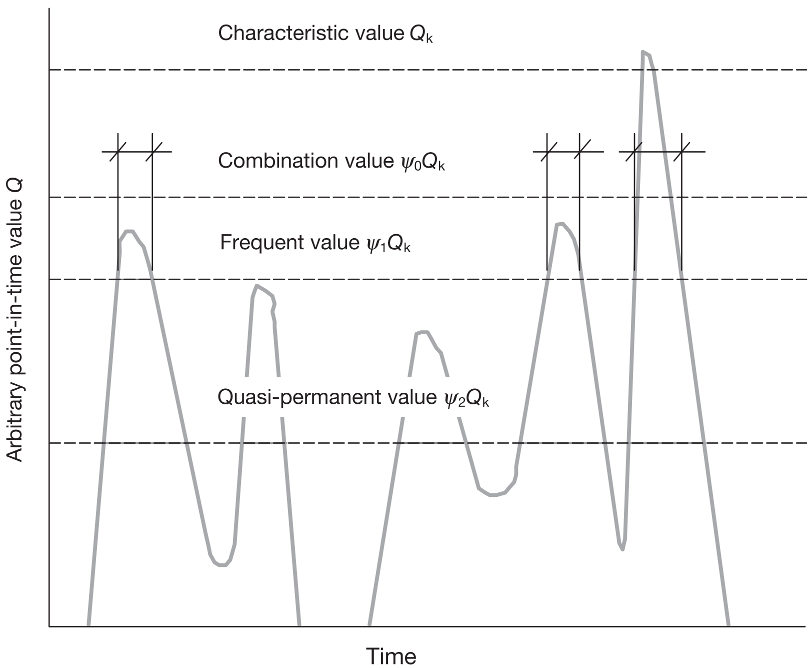


Figure 1.1. Representative values

A variable action has the following representative values (see Figure 1.1);

- the characteristic value, Q_k;
- the combination value, $\psi_0 Q_k$;
- the frequent value, $\psi_1 Q_k$;
- the quasi-permanent value, $\psi_2 Q_k$.

The combination value, $\psi_0 Q_k$, takes account of the reduced probability of simultaneous occurrence of the most unfavourable values of several independent variable actions. It is used for the verification of ultimate limit states and irreversible serviceability limit states. The frequent value, $\psi_1 Q_k$, is used for verification of ultimate limit states involving accidental actions and reversible limit states. The quasi-permanent value, $\psi_2 Q_k$, is also used for ultimate limit state verification involving accidental actions and for reversible serviceability limit states. The recommended values of ψ_0, ψ_1, ψ_2 for buildings are shown in Table 1.3 (reproduced from EN 1990).

Table 1.3. ψ factors for buildings

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A: domestic, residential areas	0.7	0.5	0.3
Category B: office areas	0.7	0.5	0.3
Category C: congregation areas	0.7	0.7	0.6
Category D: shopping areas	0.7	0.7	0.6
Category E: storage areas	1.0	0.9	0.8
Category F: traffic area, vehicle weight ≤ 30 kN	0.7	0.7	0.6
Category G: traffic area, 30 kN < vehicle weight ≤ 160 kN	0.7	0.5	0.3
Category H: roofs	0[1]	0	0
Snow loads on buildings (see EN 1991-1-3)			
Finland, Iceland, Norway, Sweden	0.70	0.50	0.20
Remainder of CEN Member States, for sites located at altitude H > 1000 m A.S.L.	0.70	0.50	0.20
Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m A.S.L.	0.50	0.20	0
Wind loads on buildings (see EN 1991-1-4)	0.6[2]	0.2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0.6	0.5	0

[1] 0.7 in the UK national annex.
[2] 0.5 in the UK national annex.

Information on combining Actions for particular design situations is given in the Chapter on EN 1990 for both ultimate and serviceability limit states.

EN 1991-1-1 *Eurocode 1: Part 1.1: Densities, self-weight and imposed loads*

Scope and field of application

EN 1991-1-1 covers the assessment of actions for use in structural design due to:

a) the density of construction materials and stored materials;
b) the self-weight of structural elements and whole structures and some fixed non-structural items,
c) imposed loads on floors and roofs of buildings (but excluding snow, which is covered by Part 1.3).

Densities of construction and stored materials

Differences in the national codes of the CEN Member States imposed constraints on the development of this part of Eurocode 1. It was not possible to describe the densities of the construction or stored materials as either mean or characteristic values since both of these terms imply some understanding of the underlying statistical distribution of the densities. They are therefore called nominal densities but can be treated as characteristic values when determining self-weights. EN 1991-1-1, in its Annex A, gives comprehensive tables for densities of construction and stored materials, which are therefore described as nominal values. For materials where the bulk weight density has significant variability according to its source, a range of values is provided. As an example, 'Table A.4 Construction materials-metals' is shown in Table 1.4.

The guidance given in EN 1991-1-1 on densities is generally similar to that contained in BS 648 which will be withdrawn.

Self-weight of structural elements

Methods are provided for assessing the self-weight of construction elements in buildings, e.g. floors, walls, partitions, roofs, cladding, finishes and fixed services.

Table 1.4. Table A.4 – Construction materials-metals (from EN 1991-1-1)

Materials	Density γ (kN/m^3)
Metals	
Aluminium	27.0
Brass	83.0 to 85.0
Bronze	83.0 to 85.0
Copper	87.0 to 89.0
Iron, cast	71.0 to 72.5
Iron, wrought	76.0
Lead	112.0 to 114.0
Steel	77.0 to 78.5
Zinc	71.0 to 72.0

As in BS 6399-1, loads due to movable partitions are treated as imposed loads but in a slightly different way and there is no minimum load on floors of offices.

Provided that a floor allows a lateral distribution of loads, the self-weight of movable partitions may be taken into account by a uniformly distributed load, q_k, which should be added to the imposed loads of floors. This defined uniformly distributed load is dependent on the self-weight of the partitions as follows:

- for movable partitions with a self-weight ≤ 1.0 kN/m wall length: $q_k = 0.5\,\text{kN/m}^2$;
- for movable partitions with a self-weight ≤ 2.0 kN/m wall length: $q_k = 0.8\,\text{kN/m}^2$;
- for movable partitions with a self-weight ≤ 3.0 kN/m wall length: $q_k = 1.2\,\text{kN/m}^2$.

For both traffic and railway bridges, the determination of the self-weight of construction elements including coating, services and other non-structural elements is also explained. To determine the upper and lower characteristic values of self-weight of waterproofing, surfacing and other coatings for bridges, where the variability of their thickness may be high, a deviation of the total thickness from the nominal or other specified values should be taken into account for which recommended values are given.

Imposed loads on buildings

Table 4 of EN 1991-1-1 gives characteristic values of loads for floors and roofs for the following categories of occupancy and use:

- residential, social, commercial and administration areas;
- garage and vehicle traffic;
- areas for storage and industrial activities including actions induced by forklifts and other transport vehicles;
- roofs;
- helicopter landing areas;
- barriers and walls having the function of barriers.

A reduction factor α_A may be applied to the q_k values for imposed loads given in EN 1991-1-1 for floors, and accessible roofs for maintenance purposes. α_A is a function of ψ_0 and the floor area A.

Additionally, provided that the area is classified according to EN 1991-1-1 6.1 into the categories residential, social, commercial and administration areas (categories A to D as described in Table 4 of EN 1991-1-1), for columns and walls the total imposed loads from several storeys may be multiplied by the reduction factor α_n (where α_n is a function of ψ_0 and the number of stories n).

The UK national annex does not allow the use of these reduction factors and specifies the use of the reduction factors in BS 6399-1. The difference in reduction factors α_A and α_n are shown in Tables 1.5 and 1.6 respectively which are reproduced from *Designers' Guide to Eurocode 1: Actions on Buildings* [1].

The basis for the determination of the characteristic loads is given by Gulvanessian, Formichi and Calgaro [1].

Table 1.5. Reduction factor α_A for floors (EN 1991-1-1 versus national annex)

A (m²)	α_A (EN 1991-1-1) $\psi_0 = 0.7$	α_A (UK national annex for EN 1991-1-1)
40	0.75	0.96
80	0.63	0.92
120	0.60	0.88
160	0.60	0.84
240	0.59	0.76

Table 1.6. Reduction factor α_n for imposed loads from several storeys (EN 1991-1-1 versus national annex)

n	α_n (EN 1991-1-1) $\psi_0 = 0.7$	α_n (UK national annex for EN 1991-1-1)
1	1	1
2	1	0.9
3	0.9	0.8
4	0.85	0.7
5	0.82	0.6
6	0.8	0.6
7	0.79	0.6
8	0.78	0.6
9	0.77	0.6
10	0.76	0.6

Implications for practice in the UK

The scope of BS EN 1991-1-1 is greater than for the appropriate UK national codes (BS 6399-1 and BS 648). There remain some topics (e.g. vertical loads on parapets and values for actions for storage and industrial use) which are not covered as comprehensively in BS EN 1991-1-1 when compared to BS 6399, and these topics will feature in a complementary document published by BSI, PD 6688-1-1, which will also provide background information.

Most of the characteristic values for imposed loads are given in ranges, with a value recommended within the range, and the UK national annex generally specifies the values given in BS 6399-1. EN 1991-1-1 used together with the national annex will not alter current practice in the UK.

EN 1991-1-2 *Eurocode 1: Part 1.2: Actions on structures exposed to fire*

Scope and field of application

EN 1991-1-2 covers the actions to be used in the structural design of buildings and civil engineering works where they are required to give adequate performance in fire exposure. It is intended that EN 1991-1-2 is used with

EN 1990 and with the Parts on structural fire design in Eurocodes 2 to 6 and 9 [4], [5] and [6]. For fire design, fire actions are the dominant action.

EN 1991-1-2 provides general guidance and actions for the structural design of buildings exposed to fire. In addition to the foreword and Section 1 'General', EN 1991-1-2 [4] contains the following main sections:

- Section 2 Structural fire design procedure;
- Section 3 Actions for temperature analysis (thermal actions);
- Section 4 Actions for structural analysis (mechanical actions);
- Annex A (informative) Parametric temperature/time curves;
- Annex B (informative) Thermal actions for external members – simplified calculation method;
- Annex C (informative) Localized fires;
- Annex D (informative) Advanced fire models;
- Annex E (informative) Fire load densities;
- Annex F (informative) Equivalent time of fire exposure;
- Annex G (informative) Configuration factor.

The Parts 1–2 of material Eurocodes EN 1992 to EN 1996 and EN 1999 that deal with passive fire protection of construction works made of different materials represent an extension of the basic document EN 1991-1-2.

Essentially the objective is to limit risk to life from fire by meeting the following performance requirements of the structure:

- to maintain load bearing function during the relevant fire exposure;
- to meet deformation criteria where the separating or protecting function of the construction may be impaired by structural deformation in the fire;
- to maintain separating function, i.e. no integrity or insulation failure during the relevant fire exposure where fire compartmentalization is required.

Basic approaches in EN 1991-1-2

As indicated in Figure 1.2, two possible methods are given in EN 1991-1-2 to determine thermal actions due to fire: the prescriptive approach and the performance-based approach.

The prescriptive approach uses nominal fires (standard temperature/time curves) to determine the thermal actions.

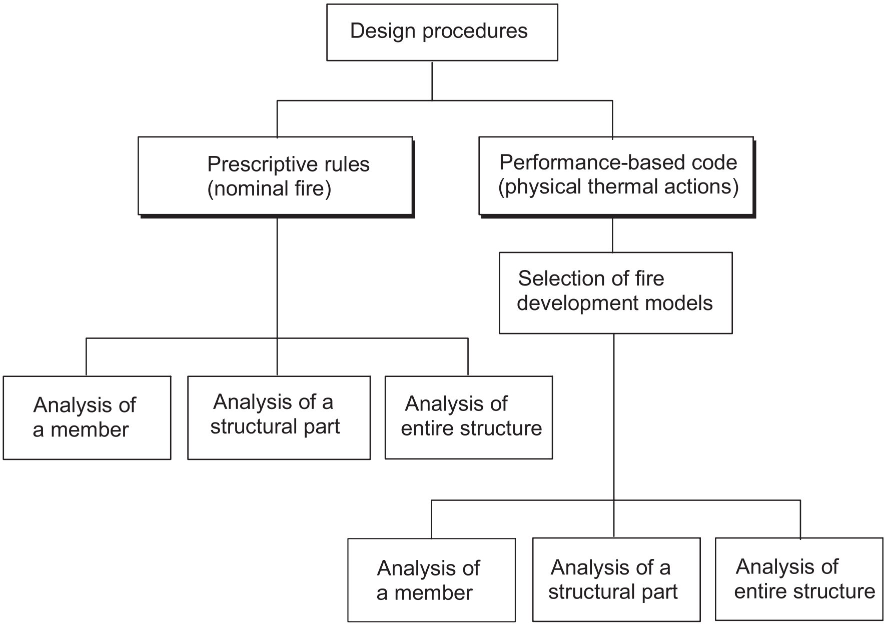


Figure 1.2. Design procedures in EN 1991-1-2

The performance-based approach, using fire safety engineering, refers to thermal actions based on physical and chemical parameters (parametric temperature/time curves). At the present time it is possible to undertake a procedure for determining adequate performance which incorporates some, if not all, of these parameters, and to demonstrate that the structure, or its members, will give adequate performance in a real building fire.

However, where the procedure is based on a nominal fire (standard temperature/time curve), the required periods of fire resistance may be specified in such a way that the features and uncertainties considered by performance-based approach described previously are taken into account (though not explicitly).

For the first time in an international standard, informative annexes provide models for more realistic calculation of thermal actions. They use so-called parametric temperature/time curves or the equivalent time of fire exposure approach.

'Parametric fire' is a general term which covers fire evolution more in line with real fires and takes into account the main parameters which influence the growth of fires. Parametric temperature/time curves therefore vary mainly with building size, type of construction, fire load and size of openings. This Part does not provide all the data needed to allow a performance-based structural fire design.

A further informative annex of this Eurocode gives guidance on the determination of fire load densities using 'the global fire safety concept'. The design value is either based on a national fire load classification or a survey of fire loads combined with partial factors to take account of fire consequences, fire frequency and active fire safety measures [6].

Design procedure

Design fire scenarios

Structural fire design involves thermal actions due to fire as well as mechanical actions. As indicated in Figure 1.2, the thermal actions may be determined using either prescriptive rules (nominal fire) or physical based rules (parametric thermal curves). Actions due to fire are classified as accidental actions and should be combined with mechanical actions using combination rules provided in EN 1990 for the accidental design situation. The combined occurrence of a fire in a building and an extremely high level of mechanical loads are assumed to be very small. Simultaneous occurrence with other independent accidental actions need not be considered. However, EN 1991-1-2 does require consideration of risks of fire in the wake of other accidental actions. Post-fire situations after the structure has cooled down are not within the scope of the document.

In accordance with clause 2 of EN 1991-1-2, a structural fire design analysis should follow these steps:

- selection of the relevant design fire scenarios;
- determination of the corresponding design fires;
- calculation of temperature evolution within the structural members;
- calculation of the mechanical behaviour of the structure exposed to fire.

Selection of the relevant design fire scenarios and corresponding design fires should be done on the basis of general principles of risk analysis taking into account possible risks due to other accidental actions. The design fire should usually be applied only to one fire compartment. Post-fire situations after the structure has cooled down need not be considered in fire design.

Temperature analysis should take into account the position of fire in relation to the structural member and separating walls. Depending on particular conditions, the analysis may be based on a nominal temperature curve without a cooling phase of full duration of the fire.

Mechanical analysis

The analysis of mechanical behaviour of a member should consider the same duration as the temperature analysis. In accordance with clause 2 of EN 1991-1-2, three design requirements should be generally verified. The time requirement is given by the inequality:

$$t_{fi,d} \geq t_{fi,requ}$$

where

$t_{fi,d}$ is the design value of the fire resistance time, and
$t_{fi,requ}$ is the required fire resistance time

Considering bearing capacity of a structural member the following condition is applied

$$R_{fi,d,t} \geq E_{fi,d,t}$$

where

$R_{fi,d,t}$ is the design value of resistance of the member in the fire situation at time t, and
$E_{fi,d,t}$ is the design value of the load effect of the relevant actions in the fire situation at time t.

In addition to the design criteria expressed by the previous expressions, in some cases the material temperature should also be checked using the condition

$$\Theta_d \leq \Theta_{cr,d}$$

where

Θ_d is the design value of material temperature, and
$\Theta_{cr,d}$ is the design value of the critical material temperature

A number of computational tools and software products are available to verify the three design conditions above.

Implementation for practice in the UK

Passive fire resistance is now fully covered by the Structural Eurocodes. The UK national annex to EN 1991-1-2 refers to a complementary document, PD 6688-1-2, which will provide background information to the national annex. Generally, the national annex will ensure that current UK practice is safeguarded.

EN 1991-1-3 *Eurocode 1: Part 1.3: Snow loads*

Scope and field of application

EN 1991-1-3 provides guidance for the calculation of:

- snow loads on roofs which occur in calm or windy conditions;
- loads on roofs which occur where there are obstructions, and by snow sliding down a pitched roof onto snow guards;
- loads due to snow overhanging the cantilevered edge of a roof;
- snow loads on bridges.

EN 1991-1-3 applies to:

- snow loads in both maritime (i.e. UK) and continental climates;
- new buildings and structures;
- significant alterations to existing buildings and structures.

It does not generally apply to sites at altitudes above 1500 m.

The basis of EN 1991-1-3 has been described elsewhere [1], [7].

The scopes of BS EN 1991-1-3 and BS 6399-2 (those relating to snow loads) are similar. However, BS EN 1991-1-3 applies to sites at altitudes to 1500 m (the limit in BS 6399-2 is 500 m).

Format for taking account of climatic variation

Both the initial deposition and any subsequent movements of snow on a roof are affected by the presence of wind. In design the general lack of data on the combined action of wind and snow is normally overcome by considering one or more critical design situations. For snow loads the load arrangement for these situations are described below.

Undrifted snow load on the roof

Load arrangement which describes the uniformly distributed snow load on the roof, affected only by the shape of the roof, before any redistribution of snow due to other climatic actions.

Drifted snow load on the roof

Load arrangement which describes the snow load distribution resulting from snow having been moved from one location to another location on a roof, e.g. by the action of the wind.

Owing to the climatic variability across Europe, EN 1991-1-3 provides different rules for 'maritime' and 'continental' weather systems. The alternatives apply for specific locations.

- For maritime weather systems where all the snow usually melts and clears between the individual weather systems and where moderate to high wind speeds occur during the individual weather system.
- For continental weather systems where the snow that falls is more persistent and where snow falling in calm conditions may be followed by further snow, carried by another weather system driven by wind and there may be several repetitions of these events before there is any significant thawing.

Maritime weather systems are associated with single snow events which occur in regions where the snow fall is considered to be associated with a weather systems of about 3 to 4 days' duration and where there is a reasonable expectation that the snow deposited on roofs will thaw between the arrival of one weather system and the next, e.g. in the UK. This situation requires the separate consideration of either uniform snow load or a drift load as the two are not expected to occur together.

Continental weather systems are associated with multiple snow events which occur where snow is more persistent and where snow falling in calm conditions may be followed by further snow, carried by another weather system driven by wind and where there may be several repetitions of these events before significant thawing. In these situations the accumulations are combined into a single load case.

It is left to the national annex to specify which should be used for a particular region.

For the two following conditions, snow loads may be treated as accidental actions.

i. Exceptional snow load on the ground in some regions, particularly southern Europe, where isolated and extremely infrequent very heavy snow falls have occurred
ii. Exceptional snow drifts (in maritime climates, e.g. UK).

Characteristic value of snow load on the ground

The snow load on the ground is that assumed to occur in perfectly calm conditions. It is usually determined from records of snow load or snow depth measured in well-sheltered areas. The characteristic value is defined as the value with an annual probability of exceedance 0.02.

In the BS EN 1991-1-3 snow map, the UK is divided into zones. An expression is given to determine the snow load on the ground which depends upon the zone number and the altitude of the site. This is different to the snow map in BS 6399 in which the snow load on the ground is determined through isopleths.

Method of assessment of snow load on the roof

The snow load on the roof is determined by multiplying the characteristic value of the snow load on the ground by a snow load shape coefficient μ. The snow load on the roof is affected by the topography of the site and the amount of heat loss through the roof and EN 1991-1-3 makes provision for adjustment of the roof snow load using an exposure and thermal coefficient factors. Thus snow loads on roofs for the persistent/transient design situations are determined as follows:

$$s = \mu_i C_e C_t s_k$$

where:

μ_i is the snow load shape coefficient
s_k is the characteristic value of snow load on the ground
C_e is the exposure coefficient
C_t is the thermal coefficient

Snow load shape coefficients

The different snow load coefficients to be considered in design relate to different climatic conditions (maritime and continental) and are given for both the un-drifted and the drifted load arrangements.

EN 1991-1-3 provides shape coefficients for mono-pitch, duo-pitched, multi-pitched and cylindrical roofs and coefficients for drifting at abrupt changes in roof height and at obstructions on roofs for both maritime and continental climate areas.

As an example, Figure 1.3 shows snow load shape coefficients for continental and maritime climates. In the diagrams, case (i) indicates the un-drifted load arrangement and cases (ii) and (iii) indicate the drifted load arrangements.

Implications for practice in the UK

EN 1991-1-3 is very similar to BS 6399-2 and should not provide any problems to the UK engineer. However, EN 1991-1-3 does not provide prescriptive clauses for certain small building roofs as BS 6399-2 does.

EN 1991-1-4 *Eurocode 1: Part 1.4: Wind actions*

Scope and field of application

BS EN 1991-1-4 is applicable to:

- building and civil engineering works with heights up to 200 m;
- bridges with spans of not more than 200 m (subject to certain limitations based on dynamic response criteria);
- land-based structures, their components and appendages.

The specific exclusions are:

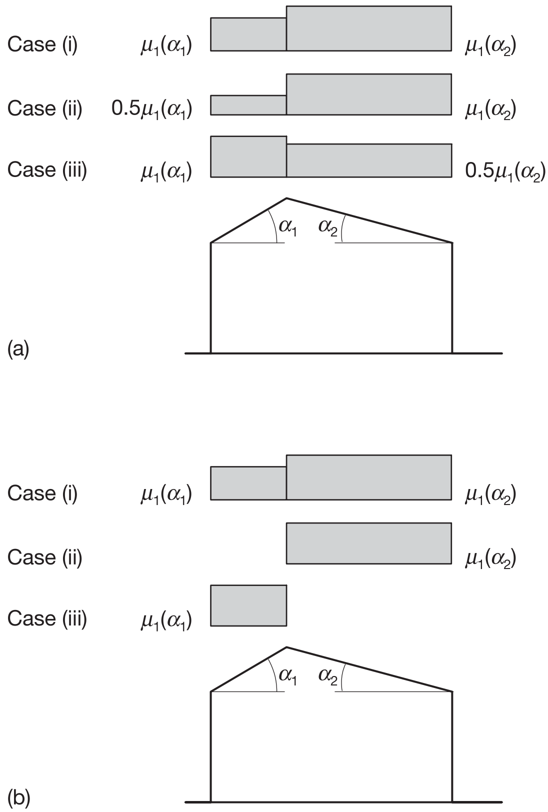


Figure 1.3. Snow load shape coefficients (a) for continental climates and (b) for maritime climates

- lattice towers with non-parallel chords;
- guyed masts and guyed chimneys;
- cable supported bridges;
- bridge deck vibration from transverse wind turbulence;
- torsional vibrations of buildings;
- modes of vibration higher than the fundamental mode.

The contents of EN 1991-1-4 are as follows:

Section 1 General
Section 2 Design situations
Section 3 Modelling of wind actions
Section 4 Wind velocity and velocity pressure
Section 5 Wind actions

Section 6 Structural factor $c_s c_d$
Section 7 Pressure and force coefficients
Section 8 Wind actions on bridges
Annex A (informative) Terrain effects
Annex B (informative) Procedure 1 for structural factor $c_s c_d$
Annex C (informative) Procedure 2 for structural factor $c_s c_d$
Annex D (informative) Graphs of $c_s c_d$ for common building forms
Annex E (informative) Vortex shedding and aeroelastic instabilities
Annex F (informative) Dynamic characteristics of structures

Modelling of wind actions

The nature of wind actions is that they fluctuate with time and act directly as pressures on the external surfaces of enclosed structures and, because of porosity of the external surface, also act indirectly on the internal surfaces. They may also act directly on the internal surface of open structures. Pressures act on areas of the surface resulting in forces normal to the surface of the structure or of individual cladding components. Additionally, when large areas of structures are swept by the wind, friction forces acting tangentially to the surface may be significant.

In EN 1991-1-4, a wind action is represented by a simplified set of pressures or forces whose effects are equivalent to the extreme effects of the turbulent wind.

The wind actions calculated using EN 1991-1-4 are characteristic values. They are determined from the basic values of wind velocity or the velocity pressure. The basic values are characteristic values having annual probabilities of exceedence of 0.02, which is equivalent to a mean return period of 50 years.

The effect of the wind on the structure (i.e. the response of the structure), depends on the size, shape and dynamic properties of the structure. EN 1991-1-4 covers dynamic response due to along-wind turbulence in resonance with the along-wind vibrations of a fundamental flexural mode shape with constant sign.

The response of structures is calculated from the peak velocity pressure, q_p, at the reference height in the undisturbed wind field, the force and pressure coefficients and the structural factor $c_s c_d \cdot q_p$ depends on the wind climate, the terrain roughness and orography, and the reference height.

Table 1.7. Terrain categories and terrain parameters

Terrain category	z_0 (m)	z_{min} (m)
0 Sea or coastal area exposed to the open sea	0.003	1
I Lakes or flat and horizontal area with negligible vegetation and without obstacles	0.01	1
II Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0.05	2
III Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0.3	5
IV Area in which at least 15% of the surface is covered with buildings and their average height exceeds 15 m	1.0	10

The terrain categories are illustrated in Annex A.1 of EN 1991-1-4.

The fundamental value of the basic wind velocity, $v_{b,0}$, is the characteristic 10 min mean wind velocity, irrespective of wind direction and time of year, at 10 m above ground level in open country terrain with low vegetation such as grass and isolated obstacles with separations of at least 20 obstacle heights (Terrain Category II, see Table 1.7). The UK national annex provides a map on this basis.

Wind velocity and velocity pressures

The basic wind velocity shall be calculated from the expression below:

$$v_b = c_{dir} c_{season} v_{b,0}$$

where

v_b is the basic wind velocity
$v_{b,0}$ is the fundamental value of the basic wind velocity, as defined above
c_{dir} is the directional factor, and
c_{season} is the season factor

The mean wind velocity $v_m(z)$ at a height z above the terrain depends on the terrain roughness and orography and on the basic wind velocity, v_b, and is determined from the expression below

$v_m(z) = c_r(z)c_o(z)$

where

$c_r(z)$ is the roughness factor, given in 4.3.2 of EN 1991-1-4, and
$c_o(z)$ is the orography factor, taken as 1.0 unless otherwise specified in 4.3.3 of EN 1991-1-4.

Ground roughness

The roughness factor, $c_r(z)$, accounts for the variability of the mean wind velocity at the site of the structure due to:

- the height above ground level;
- the ground roughness of the terrain upwind of the structure in the wind direction considered.

The determination of $c_r(z)$ is dependent upon:

- z_0 the roughness length;
- z_{min} the minimum height defined previously for the particular terrain.

EN 1991-1-4 gives z_0 and z_{min} values for the terrains which are described here in Table 1.7.

There are three terrain categories in the UK national annex:

- Terrain category 0 is referred to as sea terrain;
- Terrain categories I and II have been considered together to give a single terrain category referred to as country terrain;
- Terrain categories III and IV have been considered together to give a single terrain category referred to as town terrain.
- All inland lakes extending more than 1 km in the direction of wind and closer than 1 km upwind of the site should be treated as sea.

Terrain orography

Where orography (e.g. hills, cliffs etc.) increases wind velocities by more than 5%, the effects should be taken into account using the orography factor c_o.

The effects of orography may be neglected when the average slope of the upwind terrain is less than 3°. The upwind terrain may be considered up to a distance of 10 times the height of the isolated orographic feature.

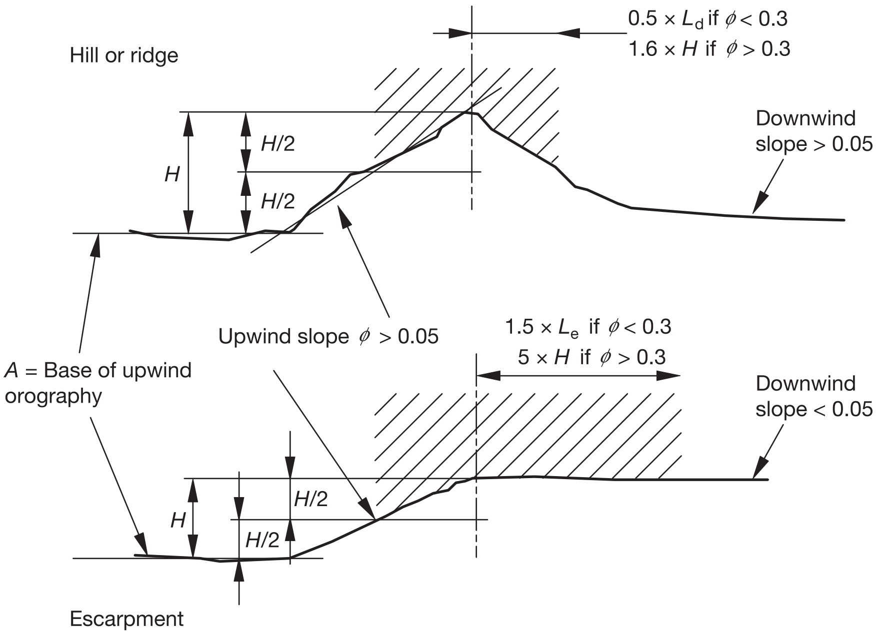


Figure 1.4. Significance of orography

The UK national annex gives a diagram (see Figure 1.4) that indicates where orography may be significant. Shaded areas show where orography is significant.

A summary of recommended calculation procedures for the determination of wind actions is given in Table 1.8.

It should be noted that procedures for determining some of the parameters (e.g. terrain category, turbulence intensity, I_v, orography coefficient, $c_o(z)$, roughness coefficient, $c_r(z)$, structural factor, $c_s c_d$) are different in the UK national annex.

The value of $c_s c_d$ may be taken as 1.0 in the following cases:

- for buildings with a height < 15 m;
- for cladding elements with a natural frequency > 5 Hz;
- for framed buildings with structural walls of < 100 m and not more than 4 times the in-wind depth;
- for circular cross-section chimneys with a height < 60 m and not more than 6.5 times the diameter.

Table 1.8. Recommended (EN 1991-1-4) calculation procedures for the determination of wind actions

Parameter	Subject reference in EN 1991-1-4
Peak velocity pressure, q_p	
Basic wind velocity, v_b	4.2 (2)P
Reference height, z_e	Clause 7
Terrain category	Table 4.1
Characteristic peak velocity pressure, q_p	4.5 (1)
Turbulence intensity, I_v	4.4
Mean wind velocity, v_m	4.3.1
Orography coefficient, $c_o(z)$	4.3.3
Roughness coefficient, $c_r(z)$	4.3.2
Wind pressures, e.g. for cladding, fixings and structural parts	
Internal pressure coefficient, c_{pi}	Clause 7
External pressure coefficient, c_{pe}	Clause 7
External wind pressure, $w_e = q_p c_{pe}$	5.1 (1)
Internal wind pressure, $w_i = q_p c_{pi}$	5.1 (2)
Wind forces on structures, e.g. for overall wind effects	
Structural factor, $c_s c_d$	6
Wind force, F_w, calculated from force coefficients	5.2 (2)
Wind force, F_w, calculated from pressure coefficients	5.2 (3)

c_s accounts for the non-simultaneous occurrence of peak pressure
c_d accounts for the vibrations of the structure due to turbulence

Wind action on bridges

Clause 8 of EN 1991-1-4 covers wind actions on bridges and the limitations are given below.

- Field of application: span length < 200 m, height above ground < 200 m.
- Road and rail bridges < 40 m span can normally be considered to be static and $c_s c_d$ may be taken as 1.0.
- Only applies to single decks (but multiple spans).
- Simplified procedure for force in x-direction

$$F_w = q_p(z_e) c_f A_{ref} (c_s c_d)$$

where

$q_p(z_e)$ is the peak velocity pressure at height z_e, and
c_f is the force coefficients.

Implications for practice in the UK

The scope of BS EN 1991-1-4 is much wider than BS 6399-2, as it includes wind actions on other structures, which in the UK are given in a number of other British Standards and design guides. In some cases, there is no equivalent UK standard, e.g. dynamic response of certain buildings. The national annex to BS EN 1991-1-4, which uses substantial information from BS 6399-2, will refer to a complementary document, PD 6688-1-4, which will give background information to the national annex and other essential advice. The effects on UK practice should prove neutral.

EN 1991-1-5 *Eurocode 1: Part 1.5: Thermal actions*

Scope and field of application

EN 1991-1-5 gives principles, rules and methods of calculating thermal actions on buildings, bridges and other structures including their structural components. Principles for determining thermal actions for claddings and other appendages on the building are also provided.

Characteristic values of thermal actions are provided for the design of structures which are exposed to daily and seasonal climatic changes. Structures in which thermal actions are mainly a function of their use (e.g. chimneys, cooling towers, silos, tanks, warm and cold storage facilities, hot and cold services) are also treated. The characteristic values of isotherms of national minimum and maximum shade air temperatures are provided in the form of maps or in other forms in the national annexes.

Underlying philosophy

The underlying philosophy of Part 1.5 is that the temperature distribution within a cross-section leads to a deformation of the element and/or, when the deformation is restrained, the occurrence of stresses in the element.

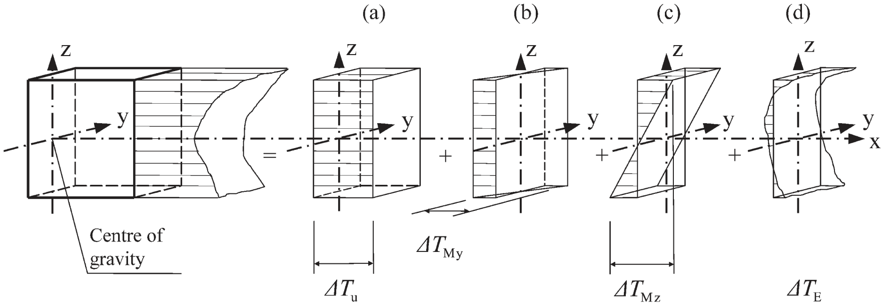


Figure 1.5. Representation of constituent components of a temperature profile

EN 1991-1-5 gives procedures for load bearing structures to be checked to ensure that thermal effects do not cause over-stressing of the structural elements, either by the provision of expansion joints or by including the effects in the design.

Classification and representation of thermal actions

Thermal actions are classified as variable free actions and are indirect actions. The characteristic values given are generally 50 year return values.

EN 1991-1-5 splits the temperature distribution within an individual structural element into the following four essential components (see Figure 1.5):

- a uniform temperature component;
- a linearly varying temperature component about the z-z axis;
- a linearly varying temperature component about the y-y axis;
- a non-linear temperature distribution.

Temperature changes in buildings

This section of EN 1991-1-5 provides general guidelines and advice on matters which should be considered.

Thermal actions on buildings due to climatic and operational temperature changes need to be considered in the design of buildings where there is a possibility of the ultimate or serviceability limit states being exceeded due to thermal movement and/or stresses.

Volume changes and/or stresses due to temperature changes may also be influenced by:

- shading of adjacent buildings;
- use of different materials with different thermal expansion coefficients and heat transfer;
- use of different shapes of cross-section with different uniform temperature.

Temperature changes in bridges

EN 1991-1-5 treats the temperature changes in bridges in very much more detail than in buildings. It groups bridge superstructures into three groups:

Group 1 Steel deck on steel box, truss or plate girders;
Group 2 Concrete deck on steel box, truss or plate girders;
Group 3 Concrete slab or concrete deck on concrete beams or box girders.

The rules provided apply to bridge decks that are exposed to daily and seasonal climatic effects. EN 1991-1-5 states that all thermal actions should be assessed by the uniform temperature component and the linear temperature components.

The characteristic value of the uniform temperature component depends on the minimum and maximum effective temperatures which a bridge will achieve over a prescribed period of time. For the three groups of bridges, guidance is provided on the determination of the minimum/maximum bridge temperatures from the minimum/maximum shade air temperatures.

Guidance is provided for determining the characteristic value of the vertical temperature component. Two approaches are provided, and either approach 1 (vertical linear component) or approach 2 (vertical temperature components with non-linear effects) may be used.

For approach 1 (the linear approach), over a prescribed period of time, heating and cooling of a bridge deck's upper surface will result in maximum positive (top surface warmer) and maximum negative (bottom surface warmer) temperature variation. Rules for determining these values for the three groups of bridges (both road and rail) and for adjusting these values to take into account varying thicknesses of surfacing are given.

For approach 2 (the non-linear approach) recommended values of vertical temperature differences for bridge decks are given in three figures, which are similar to the corresponding figures of BS 5400. In these figures 'heating' refers to conditions such that solar radiation and other effects cause a gain in heat through the top surface of the bridge deck. Conversely, 'cooling' refers to conditions such that heat is lost from the top surface of the bridge deck as a result of re-radiation and other effects.

The UK national annex to EN 1991-1-5 stipulates the use of approach 2.

Temperature changes in industrial chimneys and pipelines

EN 1991-1-5 provides quantifiable values for thermal actions in chimneys and pipelines resulting from climatic effects, due to the variation of shade air temperature and solar radiation. It requires the values of operating process temperature to be obtained from the project specification. For structures in contact with heated gas flow or heated material (e.g. chimneys, pipelines and silos) the following thermal actions are defined:

- temperature distribution for normal process conditions;
- accidental temperature distribution from failures in operation.

Characteristic values of maximum and minimum flue gas temperatures are not given. They are required to be provided in the project specification for a 50 year return period.

Implications for practice in the UK

The guidance in this part, in particular the guidance relating to building structures, is not covered in UK loading standards. The national annex to BS EN 1991-1-5 will refer to a complementary document, PD 6688-1-5, which will provide background information to the national annex. Regarding bridges, the guidance in EN 1991-1-5 and the UK national annex is very similar to current practice.

EN 1991-1-6 *Eurocode 1: Part 1.6: Actions during execution*

Scope and field of application

EN 1991-1-6 covers assessment of actions, combinations of actions and environmental influences during the execution stage, including those actions applied to auxiliary construction works, e.g. scaffolding, propping and bracing, for use in structural design of buildings and bridges.

The safety of people on construction sites is not within the scope of EN 1991-1-6.

Design situations during execution

EN 1991-1-6 gives guidance on identification of transient design situations.

EN 1991-1-6 requires that any selected transient design situation be associated with 'a nominal duration equal to, or greater than, the anticipated duration of the stage of execution under consideration'. The design situations should take into account the likelihood for any corresponding return periods of variable actions, e.g. climatic actions.

Recommended return periods of climatic actions are given depending on the nominal duration of the relevant design situation. Further information on these is given by Gulvanessian, Formichi and Calgaro [1].

Other ultimate limit states design situations, e.g. accidental and seismic situations, need to be considered.

The serviceability limit states for the selected design situations during execution need to be verified, as appropriate, in accordance with EN 1990.

The criteria associated with the serviceability limit states during execution should take into account the requirements for the completed structure.

Operations which can cause excessive cracking and/or early deflection during execution and which may adversely affect the durability, fitness for use and/or aesthetic appearance in the final stage have to be avoided.

The need to consider seismic actions is described. Accidental actions for buildings and bridges which may lead to collapse or damage during execution are described and the need to check the relevant limit states is defined.

Representation of actions

Actions during execution are classified in accordance with EN 1990, and may include:

- those actions that are not construction loads; and
- construction loads

The representation of those actions that are not construction loads are defined, i.e. self-weight, pre-stressing, intentional imposed deformations and settlements, temperature and shrinkage actions, wind, snow and water actions, and atmospheric ice loads.

Approaches for taking construction loads into account, i.e.

- site visitors and personnel with hand tools,
- non-permanent equipment,
- storage movable items,
- movable heavy machinery and equipment (e.g. cranes),
- accumulation of waste materials,
- loads from parts of structure in temporary states,

are given and specific principles and rules are included for construction loads for buildings and bridges.

Implications for practice in the UK

The guidance provided in this document does not have a UK equivalent, and thus provides new codified information for the profession.

EN 1991-1-7 *Eurocode 1: Part 1.7: Accidental actions due to impact and explosions*

Scope and field of application

EN 1991-1-7 describes safety strategies for accidental design situations. It recommends design values for the most common cases of accidental actions from impact and explosion; it gives design models and also details provisions which may be used as alternatives to design verifications. It also provides more advanced impact and explosion design concepts.

External explosion, warfare or malicious damage, or natural phenomena such as tornadoes, extreme erosion or rock falls, are not in the scope of the EN 1991-1-7.

Underlying philosophy

The selected accidental design situations should be sufficiently severe and varied so as to encompass all conditions which can be reasonably foreseen. The philosophy of Part 1.7 has been described in more detail by Gulvanessian, Formichi and Calgaro [1].

Accidental actions are required to be taken into account, depending on:

- the possible consequences of damage;
- the probability of occurrence of the initiating event;
- the provisions for preventing or reducing the hazard and the exposure of the structure to the hazard;
- the acceptable level of risk.

EN 1991-1-7 recognizes that no structure can be expected to resist all actions arising from an extreme cause and that residual risk will be present in practice. It requires there to be a reasonable probability that the structure will not be damaged to an extent disproportionate to the original cause. Localized damage due to accidental action may be acceptable.

Design situations

EN 1991-1-7 recommends two strategies to be considered for accidental design situations as follows:

- strategies based on identified accidental actions, e.g. some explosions and impact;
- strategies based on limiting the extent of localized failure.

With regard to the previous strategies, EN 1991-1-7 defines the general principles that can be used in the analysis of accidental design situations. It describes:

- the procedure for risk analysis to identify extreme events, causes and consequences;
- the safety precautions required to maintain acceptable safety by using measures to reduce the probability of the consequences of the accidental event.

Additional strategies are defined which may be used singly or in combination as measures which may be used to control the risk of accidental actions – these are based on a classification of consequences of failure as follows:

- category 1 (limited consequences) where no specific consideration of accidental actions is required;
- category 2 (medium consequences) where, depending upon the specific circumstances of the structure in question, a simplified analysis by static equivalent action models or the application of prescriptive design/detailing rules is made;
- category 3 (large consequences) where more extensive study (e.g. hazard identification, risk analysis) is required.

Accidental actions due to impact

Impact actions are defined and collision forces given for:

- impact from vehicles on walls of buildings and supporting substructures for bridges;
- impacts from vehicles on the underside of buildings and on bridge superstructures;
- impact from ships on supporting substructures;
- impact from fork lift trucks;
- impact from derailed trains;
- hard landings by helicopters on roofs.

Accidental actions due to explosions

EN 1991-1-7 covers accidental actions arising from gas explosions in buildings.

Structures classified as category 1 require no specific consideration in design of the effects of explosions. The general rules for connections and interaction between elements given in Eurocodes 2 to 9 are assumed to provide adequate safeguards.

For structures classified for category 2 and 3, Part 1.7 requires that the structure is designed to resist the accidental actions either using simplified analysis for key elements based upon equivalent static load models or by applying prescriptive design/detailing rules. For structures in category 3, the use of a risk analysis together with hazard identification is recommended.

Implications for practice in the UK

Although aspects of accidental actions are covered in BS 6399-1 and BS 5400, BS EN 1991-1-7 comprehensively covers the topic of accidental actions in one document. A categorization scheme concerning the robustness of buildings, which has also been used in Approved Document A of the Building Regulations, is from BS EN 1991-1-7 Annex A. The UK design engineer will be familiar with the design requirements of this part although risk assessments will be required for category 3 structures.

The national annex to BS EN 1991-1-7 refers to a complementary document, PD 6688-1-7, which will give background information to the national annex, in particular to risk assessments on impacts to supporting structures for bridges.

EN 1991-2 *Eurocode 1: Part 2: Traffic loads on bridges*

Scope and field of application

EN 1991-2 [8] specifies imposed loads (models and representative values) associated with road traffic, pedestrian actions and rail traffic which include, when relevant, dynamic effects and centrifugal, braking, acceleration and accidental forces. It also includes guidance on combinations with non-traffic loads on road and railway bridges, and on loads on parapets. Actions for the design of road bridges with individual spans less than 200 m and with carriageway widths not greater than 42 m are defined in EN 1991-2.

Road traffic actions and other actions specifically for road bridges

Load models

Road traffic actions are represented by a series of load models which represent different traffic situations and different components (e.g. horizontal force) of traffic action. Specific models are given for verification of fatigue.

The load models for vertical loads are as follows.

a) Load model 1: concentrated and uniformly distributed loads, which cover most of the effects of the traffic of lorries and cars. This model is intended for general and local verifications of the structure. The derivation is discussed elsewhere [9]. It is the main loading model and consists of two systems:
   - double axle concentrated loads (tandem system, TS);
   - uniformly distributed loads (UDL system), having a weight density per square metre obtained by multiplying a characteristic value by an adjustment factor.

   The UK national annex increases the applicability of the loaded lengths for this model (from 200 m to 1500 m) and gives alternative values for the uniformly distributed loads.

b) Load model 2: a single axle load applied on specific tyre contact areas which covers the dynamic effects of normal traffic on very short structural elements. This model should be separately considered and is only intended for local verifications.

c) Load model 3: a set of assemblies of axle loads representing special heavy vehicles (e.g. for industrial transport) which may travel on routes permitted for abnormal loads. This model is intended to be used only as required by the client, for general and local verifications of the structure.

   The UK national annex specifies the special vehicles currently used in UK practice.

d) Load model 4: a crowd loading. This model should be considered and is intended only for general verifications of the structure.

EN 1992-2 also provides advice on dispersal of concentrated loads, horizontal forces, braking and acceleration forces and centrifugal forces.

The various load models are combined into five groups which are then combined with non-traffic loads (e.g. climatic actions) acting on the bridge.

Fatigue load models

For verifications of resistance to fatigue, five fatigue load models of vertical forces are provided for use depending on the verification level selected from the relevant Eurocodes 2–9.

The UK national annex gives alternatives to fatigue load model 4 and restrictions to use for fatigue load model 2.

Accidental actions

The section on road bridges also provides guidance on:

- accidental actions including:
  - actions from vehicles on the bridge;
- action on parapets;
- load models on embankments.

Pedestrian, cycle actions and other actions specifically for footbridges

Vertical load models and representative values for pedestrian and cycle traffic are defined using the following load models:

- load model number 1: uniformly distributed load;
- load model number 2: concentrated load (10 kN recommended);
- load model number 3: service vehicle.

The load models are combined into two groups which are then combined with non-traffic loads (e.g. climatic actions) acting on the bridge.

Rail traffic actions and other actions specifically for rail bridges

EN 1991-2 covers the static effects of standard rail traffic operating over the standard-gauge or wide-gauge European mainline-network [2], [10].

The load models are not deemed to describe the real loads: they have been defined so that their effects, with a dynamic magnification taken into account separately, represent the effects of real traffic.

Non-accidental actions due to rail traffic are given for:

- vertical loads: four load models LM 71, LM SW (associating two sub-models SW/0 and SW/2), 'unloaded train' and HSLM (high speed load model);

- vertical loads for embankments and for earth pressure (for the sake of simplicity concentrated or linear vertical loads are replaced by distributed loads);
- dynamic effects;
- centrifugal forces;
- nosing force;
- acceleration and braking forces;
- aerodynamic effects as a result of passing trains;
- load effects from catenaries and other overhead line equipment attached to the structure.

For the consideration of centrifugal forces, EN 1991-2 recognizes that heavy traffic does not operate at high speeds whereas high speed passenger trains have light axle loadings. Centrifugal forces therefore depend on the loaded length of the bridge and on the maximum permissible speed.

Accidental actions arising from derailment on bridges are required to be taken into account so that the damage to the bridge is limited to a minimum. Prevention of overturning or collapse of the bridge is a design requirement. Other accidental actions which arise from severance of overhead line equipment and from road traffic must also be considered in design.

Fatigue damage assessment is required for all elements subjected to fluctuations of stress. Details of the service trains and traffic mixes and the dynamic enhancement are given. Each of the mixes is based on an annual traffic tonnage of (25×10^6) tonnes. Fatigue assessment is for a life of 100 years. Alternatively, the relevant authority may specify a different life and traffic mix.

The deformations and vibrations caused by the passage of rail traffic have to be limited for safety and passenger comfort.

Guidance is provided on all the topics mentioned.

Implications for practice in the UK

The national annex to BS EN 1991-2 will refer to a complementary document, PD 6688-2, which will give background information to the national annex.

EN 1991-3 *Eurocode 1: Part 3: Actions induced by cranes and machinery*

Scope and field of application

EN 1991-3 specifies actions, self-weights and imposed loads (models and representative values) associated with hoists, crabs and cranes on runway beams and static and dynamic actions induced in supporting structures by machinery.

Actions induced by cranes

Actions induced by cranes comprise actions from hoists, crabs and cranes on runway beams. Crane supporting structures are divided into two categories:

- underslung trolleys on runways;
- overhead travelling cranes.

The methods prescribed are compatible with the provisions of EN 13001-1 [11] to facilitate the exchange of data with crane suppliers. The background to the methods has been described elsewhere [12].

Actions induced by cranes are classified as variable and accidental actions and are represented by various models.

Guidance for the determination of the following load arrangements is provided:

- vertical loads from monorail hoist blocks underslung from runway beams;
- horizontal loads from monorail hoist blocks underslung from runway beams;
- vertical loads from overhead travelling cranes;
- horizontal loads from overhead travelling cranes;
- multiple crane action.

When not supplied by the crane manufacturer:

- dynamic amplification factors are given for vertical loads and advice is included on treating wind actions for cranes located outside buildings;
- dynamic amplification factors and a method of calculating the drive force on a driven wheel are given. Guidance is included on obtaining horizontal

loads and the guide force caused by skewing and the horizontal force caused by acceleration or deceleration of the crab.

Advice is also included on taking account of temperature effects in the determination of loads on access walkways, stairs, platforms and guard rails. Guidance is given on accidental actions due to buffer forces related to crane movement and movements of the crab and tilting forces. Fatigue damage equivalent loads are used to classify fatigue actions in relation to a load effect history parameter according to EN 13001-1.

Actions due to machinery

For machinery, the advice is limited to structures supporting rotating machines which induce dynamic effects in one or more planes.

Actions induced by machinery are classified as permanent, variable and accidental.

Permanent actions during service include the self-weight of all fixed and movable parts and static actions from service.

Variable actions from machinery during normal service are dynamic actions caused by accelerating masses.

Accidental actions, which may be considered to occur, are those due to accidental magnification of the eccentricity of masses, short circuit or lack of synchronization between generators and machines and impact effects from pipes on shutting down.

Guidance is given on the calculation of characteristic values of actions for normal service and accidental conditions and on determining movements caused by dynamic forces.

EN 1991-4 *Eurocode 1: Part 4: Actions in silos and tanks*

Scope and field of application

EN 1991-4 [13] gives general principles and rules for determining actions arising from the storage of bulk materials and liquids in silos and tanks. The scope is restricted to:

- silos with limited eccentricity of inlet and outlet, with small impact effects caused by filling, and with discharge devices which do not cause shock or eccentricities beyond the given limitations;
- silos containing particulate materials which are free-flowing and have a low cohesion;
- tanks with liquids stored at normal atmospheric pressure.

Background

Initial studies of codes and recommendations found mainly in Member States covering loads in silos and tanks led to work initiated by ISO being adopted in 1987 as the starting point for the preparation of Part 3 [14]. The classical Janssen theory for filling was adopted as a basis. For discharge and special cases, empirical parameters were used.

Design situations

The design situations to be considered include maximum possible filling and, where appropriate, accidental actions and situations arising from explosions, vehicle impact, seismic actions and fire.

Advice is given on measures which may be used to limit or avoid potential damage from dust explosions. Prevention of dust explosions by choice of proper maintenance and cleaning, use of safe electronic equipment and careful use of welding is advocated. Limitation of concrete cracking at the serviceability limit state is required for silos to be used for storage of water sensitive materials. Consideration of fatigue is required where the silo or tank is subjected to more than one load cycle per day.

The selection of structural form to give low sensitivity to load deviations is given as a principle and loads due to particulate materials have to be calculated for filling and discharge. Attention is drawn to the inherent variability of stored materials and the simplifications in the load models. Rules are given for calculating storage loads due to particulate materials in tall silos, squat silos and homogenizing silos and silos with a high filling velocity. Particulate material properties are obtained by a simplified approach which takes account of horizontal/vertical pressure ratio and coefficient of wall friction, or by testing.

Conclusions

EN 1991 *Eurocode 1, Actions on Structures* gives comprehensive information on all actions that should normally be considered in the design of building and civil engineering structures. It is intended primarily for use with Eurocodes 2–9 for structural design and verification on the basis of the overall principles for limit state design given in EN 1990 *Basis of Structural Design*.

References and recommended reading

[1] Gulvanessian H., Formichi P. and Calgaro J.A. *Designers' Guide to Eurocode 1: Actions on Buildings*, Thomas Telford, London, 2009
[2] Calgaro J.A., Tchumi M. and Gulvanessian H. *Designers' Guide to Eurocode 1: Actions on Bridges*, Thomas Telford, London, 2010
[3] Ostlund L. Actions. Conference, Delft, IABSE Reports, volume 74, 1996.
[4] Twilt L. and Kersken-Bradley M. EC1: Actions on structures exposed to fire. Conference, Davos, IABSE Reports, volume 65, 1992
[5] Schleich J.B. Eurocode 1: Part 2.2: Actions on structures exposed to fire. Conference, Delft, IABSE Reports, volume 74, 1996
[6] Twilt L. et al Input data for the natural fire design of building structures. Conference, Delft, IABSE Reports, volume 74, 1996
[7] Sanpaolesi L. 'Snow loading: Scientific basis, problems and challenges', *Progress in Structural Engineering and Materials*, 1998, volume 1 (4), pp443-451
[8] Calgaro J.A. 'Loads on bridges', *Progress in Structural Engineering and Materials*, 1998, volume 1 (4), pp452-461.
[9] Bruls A., Calgaro, J.A., Mathieu H. and Prat M. ENV 1991: Part 3: The main models of traffic loads on road bridges. Conference, Delft, IABSE Reports, volume 74, 1996
[10] Gandil J., Tshumi M.A., Delorme F. and Vognier P. Railway traffic actions and combination with other variable actions. Conference, Delft, IABSE Reports, volume 74, 1996
[11] EN 13001 *Cranes – Safety – Design – General – Part 1: General principles and requirements. Part 2: Load effects.*
[12] Sedlacek G. and Grosmann D. EC1: Actions induced by cranes. Conference, Delft, IABSE Reports, volume 74, 1996
[13] Rotter, J.M. 'Metal silos', *Progress in Structural Engineering and Materials*, 1998, volume 1 (4), pp428-435
[14] Nielsen J. et al. EC1: Silos and tanks. Conference, Davos. IABSE Reports, volume 65, 1992

Eurocode 2: Design of concrete structures

Owen Brooker, The Concrete Centre

Introduction

The transition to using the Eurocodes is a daunting prospect for engineers, but this needn't be the case. Industry has worked hard to ensure that there are numerous resources available, designed to assist engineers with the transition process. The key to getting into Eurocode 2 is to understand how it is set out and how it is different from existing standards. This article covers these topics and explains how Eurocode 2 fits in with other European standards. The good news is that most of the engineering principles are the same as the existing British Standards. For existing engineers is should not be like learning to drive, more like getting into a strange car for the first time – the controls are all there but it takes a few miles to learn where they are and how they operate.

A standard or a handbook?

The first difference that British engineers will come across is that Eurocode 2 does not appear to tell you how to carry out design. In Europe standards are very much seen as setting out the rules, the application of those rules is left to authors of text books or design guidance. In many ways British Standards are far more like design handbooks and provide far more practical material than other Europeans would expect to see in their standards. This why British engineers find that it does not provide the information they are expecting. A good example of this is the design expression for flexure. In British Standards we are used to being given the expressions for the lever arm, areas of steel and limiting values for K. These are not given explicitly in Eurocode 2 and in fact there appears to be very little guidance on flexural design, other than the stress blocks that can be used. However, help is at hand because this information is available in text books, and in the design guides that have been prepared by The Concrete Centre.

A stand alone document?

It is important to realise that the Eurocodes are supported by a number of European Standards; these are not themselves designated as Eurocodes, but nonetheless are required to enable the Eurocodes to be used. Figure 2.1 shows how Eurocode 2 fits into the Eurocode system.

The first point to note is that Eurocode 2 has four Parts. The main Part is Part 1.1 and this gives general rules for all structural concrete and specific rules for buildings. Often it is this part that is being referred to when citing 'Eurocode 2'.

Part 1-2 covers structural fire design and at first it seems a bit imposing to have over 100 pages on fire design when a few tables sufficed in BS 8110. However, the good news is that this Eurocode does have simplified tables in Section 5 and so designers will continue to be able to check that their design meets the requirements for fire resistance with a quick reference to a table. The remainder of the document is devoted to simplified and advanced calculation methods. The former may be useful to the structural engineer for specific situations, for example a column with an eccentric load. However, advanced methods are probably the reserve of fire engineering specialists.

Bridge designers will need to refer to Part 2, which describes how Part 1.1 should be amended to suit bridge designs. This sounds simple in theory, but in practice it is rather complicated to work out which clauses are deleted, which are amended and which are new clauses, especially when the national annexes to both documents are factored in.

The final Part of Eurocode 2 is Part 3, which covers the design of water containing structures. Again, it modifies Part 1.1, but in practice it includes additional clauses to aid the design of liquid containing structures and so it is far simpler to apply.

Designers of concrete structures will need to determine the actions on their structures and these are given in Eurocode 1. They will also need to determine

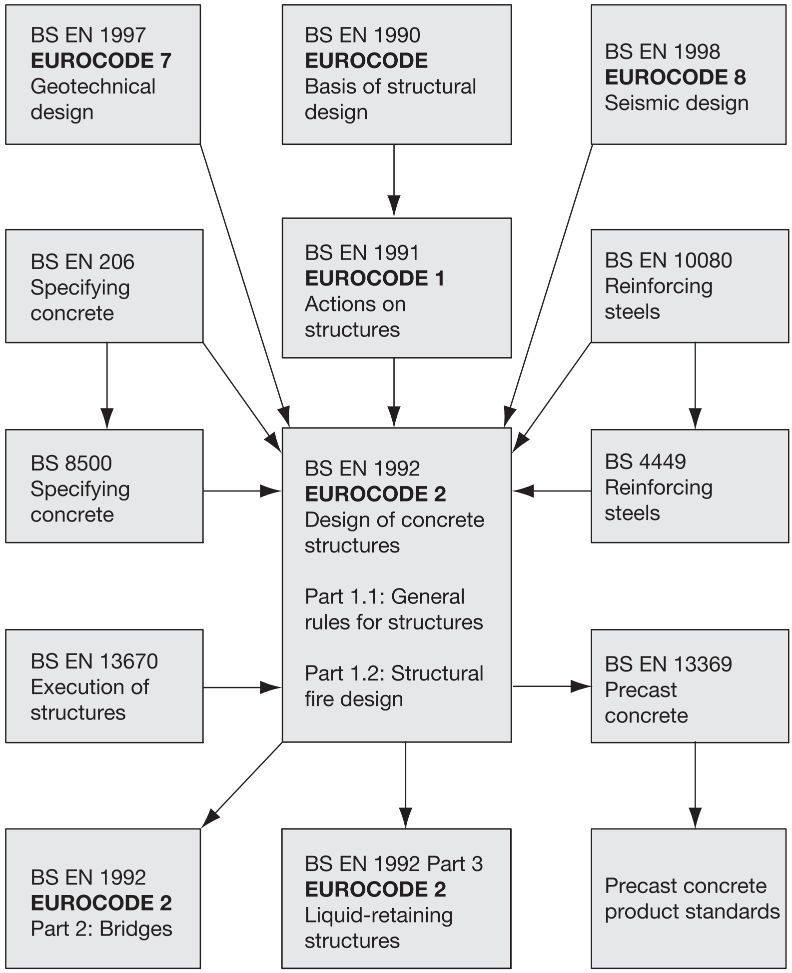


Figure 2.1. Relationship between Eurocode 2 and other European standards

the combination of these actions, and these are given in Eurocode (often incorrectly called Eurocode 0).

Any structure has to have foundations, and as these are usually concrete the concrete designer will also have to refer to Eurocode 7 to determine the load carrying capacity of the ground.

Occasionally in the UK there is the need to design for seismic conditions and for the first time this is covered by a UK standard – Eurocode 8.

Supporting European Standards

Moving on from the links between Eurocodes to look at other European Standards, UK designers should already be familiar with most of them. Specification of concrete is covered in BS 8500, which is the UK application document for BS EN 206-1. This document was originally published in 2002 and so designers should be familiar with its contents already. It brought a new approach to determining the durability resistance of concrete based on the deterioration processes.

At this point it should be noted that the BS 8500 approach to durability does not follow the approach in Eurocode 2, rather it is cited in the UK national annex to Part 1.1.

A key change in BS 8500 was the introduction of the dual classification system where the concrete cylinder strength is given alongside the equivalent concrete cube strength.

Other standards which UK designers should already have adopted are BS 4449 and BS 8666, which cover the specification and detailing of reinforcement. Here the key change is that the characteristic strength of reinforcement supplied in the UK is 500 MPa; previously the minimum strength was 460 MPa. To ensure that there was no confusion with the older steel grades the 500 grade steel is designated with an 'H'.

There is a suite of European products standards for precast concrete elements, e.g. hollowcore units and double-tee units. The over-arching standard is BS 13369, which sets out the requirements for all precast concrete products such as tolerances, durability and design.

The final piece of the jigsaw is BS EN 13670 which is the execution standard, execution being the European term used to cover construction. At the time of writing this publication has been finalized and is due for publication in the UK in time for the March 2010 transition date. The document replaces Section 6 of BS 8110 which covered workmanship and has been written to tie-in with Eurocode 2. A UK national annex will be published alongside the standard. Standard specifications, such as the *National Structural Concrete Specification* (NSCS) and *National Building Specification* (NBS) will be updated to incorporate the changes.

Non-Contradictory Complementary Information

Non-Contradictory Complementary Information (NCCI) is a specific term in the Eurocodes that refers to documents that may be used alongside a specific Eurocode and which do not contradict that Eurocode. NCCI are listed in the relevant national annex only. The two key NCCI in the UK for concrete design are PD 6687 *Background Paper to the UK national annexes to BS EN 1992-1* and PD 6687-2: *Recommendations for the design of structures to BS EN 1992-2*. 'PD' stands for published document and are published by BSI. The two documents have been prepared by the relevant BSI committee and provide background information to the choices made for the UK national annexes. They also give useful guidance and designers will find that they are almost essential to enable them to use Eurocode 2.

How is Eurocode 2 different?

The question that is often asked is, 'What are the differences between British and European Standards?' There are differences of course, but engineers should not overlook the similarities and in this chapter we will consider both.

Eurocode 2 is laid out on the basis of the phenomenon (e.g. flexure, shear, deflection) and not by element type (e.g. beam, flat slab). This makes the code more useful and engineers are not straitjacketed by thinking only in terms of the type of element. As we make more use of computer-aided engineering to be more creative in our designs, treating elements strictly as say beams or columns may not be appropriate.

Eurocode 2 uses concrete cylinders strengths, rather than cube strengths as the basis for the design calculations. This is not, as many think, because cylinders are used in continental Europe, but rather that cylinders have been used in the laboratories when testing samples and have therefore been used to develop design rules. It therefore makes more sense to use them as the basis for design. Testing of cubes can still be used to monitor concrete strength as Eurocode 2 gives the equivalent cube strengths.

In Eurocode 2 the units of stress are mega-Pascals (MPa), although interestingly Eurocode 6 still uses Newtons per square millimetre (N/mm^2).

In line with European conventions, the decimal marker is denoted by a comma, rather than a full stop. This is not necessarily a problem, provided that it is recognized by the reader. British Engineers can continue to use the full stop in their workings, but may like to consider avoiding the use of the comma as a thousands separator.

There is a new symbol that UK engineers may not be familiar with – '‰', which means a thousandth.

Designers will also find that there is a new method of presenting equations (known as 'expressions' in Eurocodes). For example, minimum cover is:

$c_{\text{min}} = \max\{c_{\text{min,b}}; c_{\text{min,dur}}; 10 \text{ mm}\}$

which means that c_{min} is the maximum of $c_{\text{min,b}}$ (minimum cover for bond) $c_{\text{min,dur}}$ (minimum cover for durability) and 10 mm.

Designers should note that the effect of geometric imperfections should be taken in addition to any lateral loads. There is no longer the philosophy of checking for either notional horizontal loads or actual lateral loads.

Eurocode 2 allows the designer to use high strength concrete, up to a class C90/105 (or class C70/85 for bridges). Above a class C50/60, the engineer will find that there are restrictions placed in Eurocode 2. For instance, there are lower strain limits, additional requirements for fire resistance and in the UK the resistance to shear should be limited to that of a class C50/60 concrete.

On the subject of shear, Eurocode 2 uses the variable strut inclination method. It is assumed that shear is resisted by a concrete strut and the shear links and longitudinal steel are acting in tension to form a truss. The designer can vary the concrete strut angle in the truss (whereas in BS 8110 and BS 5400 it was fixed at 45°). This has the advantage of including more shear links in the truss and so reduces the reinforcement requirements.

Still looking at shear, punching shear checks are based on 2*d* away from the column face and the perimeters have rounded corners.

Prestressed design is not treated as a separate process and the design expressions are written to include the effects of prestressing where appropriate.

For the determination of the anchorage and lap lengths the designer will find that Eurocode 2 is not restrictive and could enable savings to be made by applying all of the benefits. Conversely, the designers will need to work out how to rationalize the calculations for typical situations or refer to design aids where this has already been done.

Subscript	Definition
A	Accidental situation
c	Concrete
d	Design
E	Effect of action
fi	Fire
k	Characteristic
R	Resistance
w	Shear reinforcement
y	Yield strength

Figure 2.2. Selected Eurocode 2 subscripts

Perhaps the biggest challenge is the new symbols, but they will become familiar over time. The Eurocodes follow some clear guidelines for use of symbols and so there is consistency, which assists the learning process. Figure 2.2 provides some typical subscripts found in Eurocode 2 and once they are familiar it is much simpler to work out the meaning of a symbol.

What has remained the same?

All the focus is usually on what has changed, but it worth considering where there are similarities. There continue to be a number of options for the stress–strain relationships for concrete, and the option of a simplified, rectangular stress block remains, although the depth of the stress block is slightly different.

Various options for analysis remain, including elastic methods, elastic analysis with redistribution, plastic analysis and non-linear behaviour. The principle that plane sections remain plane is also used.

The load arrangements used in the UK can continue to be used, i.e. maximum hogging is determined from the full load on every span, and maximum sagging is found by checking alternate spans loaded. Note however that the permanent action partial factor should be the same in every span.

There is guidance on the effective length of members and the design moment can be taken at the face of the column, not the centre.

The principle of using span-to-depth rules to control deflections remains.

Many of the detailing rules for particular elements remain the same, although there are some slight variations. For robustness, the rules for tying have not changed except that the design vertical tie force may be slightly different.

Will there be savings?

It is likely that there will be savings in materials especially in the longer term once designers are familiar with the new codes. These potential savings come from a number of sources. Depending on your perspective they can be viewed as an erosion of safety factors or a result of our increased knowledge.

Some of the savings arise from the combinations of actions in Eurocode. With the reduction for permanent actions down to 1.35 or even 1.25 from 1.4, and the variable action partial factor reduced to 1.5 from 1.6 there are savings of 5% to 10% on the loads applied to structures at the ultimate limit state. At the serviceability limit state there is a reduction in the effective partial factor for variable actions, which can be as low as 0.3 compared to 1.0 with existing standards. Designers should of course ensure that this reduction will still provide a building or structure that will meet the client's requirements – serviceability limit states are advisory not mandatory.

Within Eurocode 2 itself the key savings come from the use of the variable strut inclination method for the design of shear reinforcement, which can reduce the number of shear links required.

There are a couple of areas where the Eurocode 2 could lead to more materials being used – more cover is often required for internal situations, mainly to allow appropriate fixing tolerances and more punching shear reinforcement may also be necessary.

Overall, the general opinion is that Eurocodes will lead to more efficient concrete structures and the savings over current practice will be 5% to 10%.

What resources are available?

The Concrete Centre and other industry bodies have produced a number of resources to assist the engineer.

The Concrete Centre resources include a series of '*How to …*' guides, which are now available as books. Each guide gives a brief overview of a particular topic and describes what the Eurocodes require and how to carry out the design checks. They are succinct and enable the designer to confidently tackle the design of particular elements through the use of flow charts. They include derived expressions and design aids where these are required to carry out designs.

How to guides:

- Introduction
- Getting started
- Slabs
- Beams
- Columns
- Foundations
- Flat slabs
- Deflection calculations
- Retaining walls
- Detailing
- BS 8500 for building structures
- Structural fire design

The *Concise Eurocode 2* is designed to be an easy way for a designer to find their way into the code. The essential clauses are given along with the UK national annex values and clear references to the original clauses. There is also useful commentary, derived expressions and design aids.

The *Concise Eurocode 2 for bridges* is a similar publication for bridge engineers and is particularly useful as a starting point as it enables the designer to see at a glance what the code requires rather than deciphering it from Part 1.1 and Part 2 of Eurocode 2. The UK national annex values are included and clearly distinguished as are sections from the published documents (PDs).

The *Concrete Buildings Scheme Design Manual* is a handbook intended to assist candidates for the Institution of Structural Engineer's chartered membership examination, and has recently been updated to incorporate Eurocode 2. It includes quick design methods and design aids that have been prepared specifically for Eurocode 2 and which will be useful when preparing preliminary designs to Eurocode 2.

Economic Concrete Frame Elements has also been updated to suit Eurocode 2. It is intended to assist in the rapid sizing of typical elements for initial designs, and includes tables and charts for a variety of concrete elements including reinforced concrete, precast concrete and post-tensioned elements.

Properties of Concrete for use in Eurocode 2 provides engineers with a greater knowledge of concrete behaviour so that they can optimize the use of material aspects of concrete in their design. A series of worked examples have been prepared and will be published. These cover the analysis and design of typical elements. They are intended to show all the checks that should be satisfied for those elements, including detailing.

There are a number of other publications available from other organizations. British Precast has produced two companion guides specifically for precast concrete products entitled *Precast Eurocode 2: design manual* and *Precast Eurocode 2: worked examples*. The Institution of Structural Engineers has produced the *Manual for the design of concrete building structures to Eurocode 2* (Green book) and *Standard method of detailing structural concrete*. Thomas Telford has published *Designers guide to BS EN 1992-1-1 and BS EN 1992-1-2* and *Designers guide to BS EN 1992-2*.

When should I start using Eurocode 2?

In June 2008, BSI declared that BS 8110 was obsolescent. According to BSI, a declaration of obsolescence indicates the standard is not recommended for use in new 'equipment' but needs to be retained for the servicing of existing 'equipment' that is expected to have a long working life. For 'equipment' read 'structures'.

BSI plans to withdraw BS 8110 and other structural concrete design codes on or about 31 March 2010. 'Withdrawn' indicates that a standard is no longer current and has been superseded by another standard or is no longer relevant to industry. It will also no longer be supported by a committee, which means that it will not undergo a five-year review. The standard is not necessarily unsafe, but will increasingly become outdated and therefore not current best practice.

The implication is that Eurocode 2 should be the concrete design standard for use in the UK, and it is expected that designers will move over to using it. And with the many resources available to make the transition as easy as possible, there is no reason to feel daunted.

Eurocode 3: Design of steel structures

David Brown, Associate Director, Steel Construction Institute

Introduction

Structural engineers should be encouraged that at least in steel, design conforming to Eurocode 3 (BS EN 1993-1-1) is not significantly different to BS 5950. In fact, if the changes in presentation are stripped away, in most areas design is almost identical to both standards. This is of course to be expected, as the steel itself knows no difference, and the UK has enjoyed the benefits of mature design standards for decades – it would be very surprising if there were dramatic changes from previous UK practice. There are well-known changes:

- the nomenclature is different, with more Greek symbols;
- subscripts are important, and informative;
- the Eurocode is arranged by structural phenomena, not design routine;
- most checks are presented as expressions, not graphs or look-up tables;
- the 'simple' approaches found in BS 5950 are generally missing – the Eurocode presents the rigorous methods.

The importance of the national annex

This chapter presents an overview of the key areas of steel design to the Eurocode. In addition to highlighting the more interesting design issues, one general point about all Eurocode design should be made, which is that the national annex is crucially important. The national annex allows countries to set various parameters, but these are not limited to a modest list of factors. The national annex may also define which methods are allowed, or set limits on a range of application. In practice, the national annex often embraces the opportunity to define national parameters with enthusiasm, providing revised methods, new tables and important limitations. In the core Eurocode document, the possibility that the national annex might have an impact is identified with a Note to the clause. The Notes are easy to miss, and ignoring the impact of the national annex could be very significant. A very strong recommendation is to review each national annex, and mark the core Eurocode with the important changes. Each and every Eurocode Part has a national annex, so this is no small task. The national annex to be used is that for the country where the structure is to be built.

Loading

The loading Parts of the Eurocode are independent from the resistance Parts, but the determination of actions is nevertheless a key part of the design process. Designers are offered four ultimate limit states in BS EN 1990 – EQU, STR, GEO and FAT, covering equilibrium, strength, geotechnics and fatigue. STR will be the most relevant for building designers, where there is a choice of how the combination of factored loads (called 'the design value of the combination of actions') can be calculated:

- using expression 6.10; or
- using the most onerous of expressions 6.10a and 6.10b.

Both expressions 6.10a and 6.10b produce a lower ULS design value than 6.10, so are recommended for greatest economy. Modest experience will conclude that expression 6.10b is almost always the critical expression. Use of expression 6.10 will always be a little conservative, so can always be used.

Using expression 6.10b, members designed for vertical load only, such as beams and many columns, will be designed for 1.25 × permanent actions + 1.5 × variable actions, which is an immediate attraction of around 8% compared to BS 5950.

The stability systems will be subject to higher loads, as they must carry the lateral loads (wind) factored by 1.5, plus modest imperfection forces. This compares with 1.4 × wind in BS 5950, with no notional horizontal forces.

BS EN 1991-1-4 deserves a special mention. The national annex to this Part is a very substantial document, and should be consulted. For designers, a number of important points should be noted.

- The national annex recommends that roof coefficients be taken from BS 6399-2, not the Eurocode.
- Internal pressures may be calculated, based on opening ratios, or the two cases of +0.2 and –0.3 should be considered.
- The Eurocode comments on the common practice of so-called ‘elective’ dominant openings – those that would be dominant but are considered shut at ULS. An additional accidental case must be considered with the dominant opening.

BS EN 1990 also refers to SLS – but then directs the designer to the material standard – in this case BS EN 1993-1-1. The designer is then referred to the national annex, which in the UK simply confirms the status quo – horizontal and vertical deflections are to be checked under unfactored variable actions and permanent actions need not be included. The same familiar deflection limits in BS 5950 (loved and loathed in equal measure by those who consider them attractively vague/too definitive) reappear in the national annex.

Member resistance

In most cases, calculated resistances are close to those calculated using BS 5950, and the design process is very similar. The fundamental structural mechanics have not changed and dramatic changes in resistance should ring alarm bells. Experienced designers will have a feel for the sorts of member sizes they anticipate and that experience is equally appropriate to Eurocode designs as it is to BS 5950.

Steel strengths

Table 3.1 of BS EN 1993-1-1 provides steel strengths for thicknesses up to 40 mm, and over 40 mm. This is a good example of the subtle influence of the national annex, which is invited to allow the use of Table 3.1, or to take the steel strengths from the product standard. The UK national annex adopts the latter, so in the UK steel strengths will continue to change at thicknesses of 16 mm, 40 mm, 63 mm etc. The UK national annex also notes that for ultimate strengths, where a range is given in the product standard, the lowest value in the range should be adopted. Again this is subtle, but for S275 steels, this means that the ultimate strength, f_u, must be taken as 410 N/mm^2 rather than the more familiar 430 N/mm^2. The impact is modest, but will affect tying resistances, where ultimate strengths are used.

Steel sub-grade

Choice of steel sub-grade is very important to ensure that brittle failure does not occur. Although this issue is addressed in BS EN 1993-1-10, the strong advice is to obtain PD 6695-10 from BSI ('PD' stands for published document). The PD takes the complicated approach in the standard, and presents it is an altogether more amenable form – use of the PD is recommended. In the PD, tables of limiting thicknesses are presented for internal environments (−5 °C) and external environments (−15 °C). The limiting thickness depends on the type of fabrication (considering stress raisers and residual stresses) and the state of stress.

Imperfections

BS 5950 introduced designers to imperfections – the notional horizontal forces (NHF) are used 'to allow for the effects of practical imperfections such as a lack of verticality'. The Eurocode deals with the same issue by 'equivalent horizontal forces' (EHF). The NHF were only applied in the 'gravity load' combination, whereas unless the externally applied lateral actions are more that 15% of the vertical actions, the EHF appear in every load combination. In practice, the 15% rule means that for multi-storey frames, expect that EHF will appear. In portal frames, where the vertical loads are modest, the EHF will appear in the 'gravity' combination, but probably not in other combinations.

BS EN 1993-1-1 describes three types of imperfection:

a) frame imperfections (as discussed previously);
b) member imperfections;
c) bracing imperfections.

Frame imperfections are allowed for by the EHF, as discussed previously. The value of the EHF is given as a proportion, ϕ, of the factored vertical loads, where $\phi = \phi_0 \alpha_h \alpha_m$. The basic value of ϕ_0 is 1/200, or the 0.5% of BS 5950. Factors ϕ_h and α_m can safely be set to 1.0, but allow for the height of the structure and the number of columns that contribute to the force on the bracing system. (Eagle-eyed designers will appreciate that the definition of α_m has been taken from BS EN 1992-1-1, rather than the (inappropriate) definition in BS EN 1993-1-1.)

Designers should not worry about member imperfections. Member imperfections have always been allowed for in the member design checks found in the standard, so no change in practice is needed. Some engineers wish to design members from first principles, accounting for real material behaviour, residual stresses etc., and the Eurocode reminds such experts to allow for the inevitable member imperfections.

Bracing imperfections will be new to UK designers – but the principle is not new. If frames are imperfect, bracing systems will also be imperfect. The Eurocode allows the imperfection to be allowed for by small additional forces applied at the nodes. The result will be slightly larger forces in the bracing members. Designing for these larger forces allows for the second order effects.

Frame stability

The Eurocode approach will be familiar to UK designers. The only difference is that the check for sensitivity to second order effects is carried out under *all* the lateral loads – the externally applied loads and the EHF. Under BS 5950 the check was carried out under NHF alone. The result is almost identical, since the BS 5950 expression was a particular instance of the general expression used in BS EN 1993-1-1:

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right)$$

The fact that the wind loads are included makes little difference to the outcome, because as the horizontal action, H_{Ed}, increases, so the horizontal deflection, $\delta_{H,Ed}$, increases.

Second order effects are small enough to be ignored if $\alpha_{cr} > 10$, and if second order effects need to be allowed for, the Eurocode offers an amplifier which is identical to k_{amp} in BS 5950.

Cross-sectional resistance

The only significant change is to the shear area for rolled sections, and then the change is largely cosmetic. The shear area of a rolled section according to BS 5950 was *Dt*, but the Eurocode changes this to a rather more complex area, as illustrated in Figure 3.1.

The effect is modest, as can be seen from the typical examples in Table 3.1.

The advantageous effect of the increased shear area according to EN 1993-1-1 is somewhat offset by the formula for the shear resistance. According to the Eurocode, the shear resistance $V_{pl,Rd}$ is

$$V_{pl,Rd} = \frac{A_v (f_y / \sqrt{3})}{\gamma_{M0}}$$

which incorporates 1/√3 compared to the familiar figure of 0.6 in BS 5950.

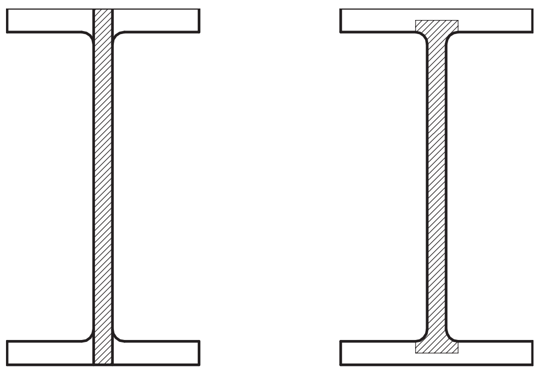


Figure 3.1. Shear areas for rolled sections in BS 5950 and BS EN 1993-1-1

Table 3.1. Examples

Section	Shear area (mm^2)		Shear resistance S275 (kN)	
	BS 5950	BS EN 1993-1-1	BS 5950	BS EN 1993-1-1
533 × 201 × 92 UKB	5072	5450	837	909
356 × 171 × 57 UKB	2900	3193	479	501
203 × 133 × 23 UKB	1158	1285	191	197

Buckling of compression members

In addition to flexural buckling, the Eurocode covers torsional buckling and torsional-flexural buckling, which are illustrated in Figure 3.2. These forms of buckling are uncommon, as they involve a bi-symmetric cruciform section or an asymmetric section used as a compression member.

The most significant change in the buckling section is the presentation of slenderness. In the Eurocode, the general expression is that the slenderness, $\bar{\lambda}$ is given by

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}}$$

where

N_{cr} is the elastic critical buckling load for the buckling mode being considered

For the common case of flexural buckling, N_{cr} is more commonly known as the Euler load, given by

$$\frac{\pi^2 EI}{L^2}$$

For flexural buckling, the calculation of N_{cr} is not the only route to determine $\bar{\lambda}$, because

$$\bar{\lambda} = \frac{\text{BS 5950 slenderness } (L / r_{yy})}{\text{a factor}}$$

which is an approach that may appeal to many designers. The factor varies with steel grade.

Having calculated the slenderness, the designer must select which curve to use (what type of member? Which axis of buckling? Etc.). The proportion of yield strength to be used when calculating the resistance (there is no direct equivalent of the compressive strength) can be found from a graph (Figure 3.3) or by calculation.

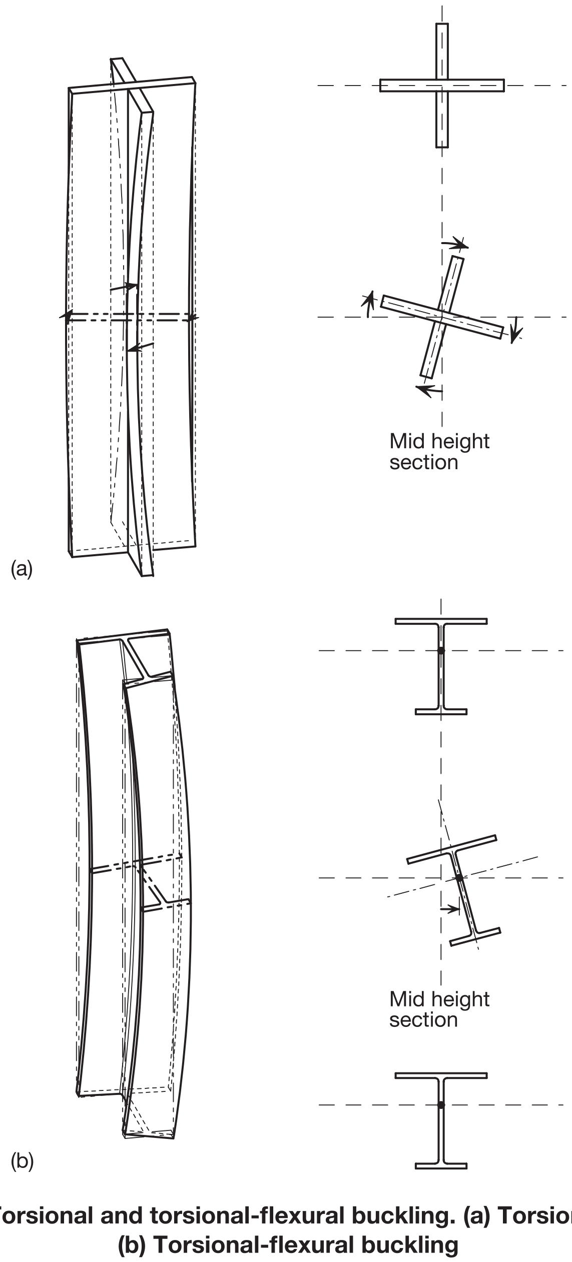


Figure 3.2. Torsional and torsional-flexural buckling. (a) Torsional buckling. (b) Torsional-flexural buckling

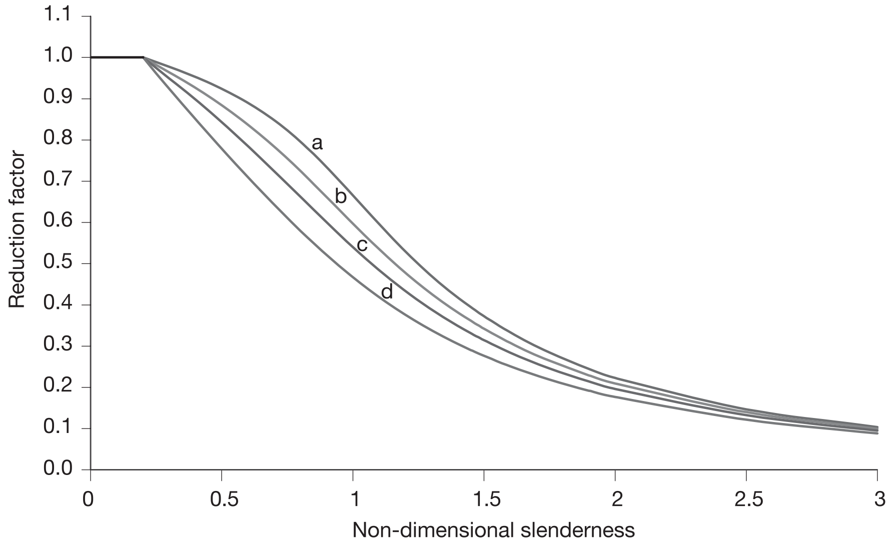


Figure 3.3. Buckling curves

Lateral torsional buckling

Having seen the presentation of flexural buckling, designers will immediately recognize the similar presentation of lateral torsional buckling. The slenderness, $\bar{\lambda}_{LT}$, is given by

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$$

in which bending terms have replaced the axial terms. One unfathomable omission from the Eurocode is any expression to calculate M_{cr}. Sources of non-conflicting complementary information (NCCI) can be used, such as http://www.access-steel.com or the useful (and free) programme *LTBeam* from CTICM in France. In BS 5950, the beneficial effects of a non-uniform bending moment diagram are accounted for by the factor m_{LT}, which is applied to the calculated buckling resistance M_b. Thus it is straightforward to prepare look-up tables which contain single values of M_b – the m_{LT} adjustment is 'outside' the calculation of M_b. The Eurocode takes a different approach and allows for the effect of a non-uniform bending moment using a C_1 factor within the calculation of M_{cr}. This means that look-up tables, such as those in the 'Blue Book' must present values of buckling resistance for different C_1 values, as shown in Table 3.2.

Table 3.2. Typical look-up tables for lateral torsional buckling

Designation Cross section Resistance (kNm) Classification	C_1 [(1)]	Buckling Resistance Moment $M_{b,Rd}$ (kNm) for Length between lateral restraints, L(m)													Second Moment of Area y-y axis I_y
		2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	cm^4
533 × 210 × 101	1.00	681	602	523	450	387	336	295	262	236	214	196	181	168	61 500
	1.50	692	692	644	581	519	462	411	367	329	297	269	250	233	
$M_{c,y,Rd}$ = 692	2.00	692	692	692	670	618	565	514	467	425	387	354	324	298	
$M_{c,z,Rd}$ = 106	2.50	692	692	692	692	689	643	598	553	511	471	435	402	373	
Class = 1	2.75	692	692	692	692	692	675	632	590	549	510	473	440	409	

The importance to the national annex can again be demonstrated within this area of the standard. The core Eurocode defines which buckling curve to use based on the *h*/*b* ratio (section height/section breadth) and proposes that an expression can be used to calculate the buckling resistance of rolled sections and 'equivalent welded sections'. The national annex is given the opportunity to influence the curves and the buckling resistance calculation. The UK national annex takes this opportunity, defining additional *h*/*b* limits. The UK national annex also specifies the constants to be used in the calculation of buckling resistance, accepting the recommended values for rolled sections, but specifying values that downgrade the resistance of welded sections.

Many designers will remember beam design according to BS 449, and look back with fondness to the simple tables in that standard. Because the physics has not changed, it is equally possible to present information in a similar way, as shown in Table 3.3.

In Table 3.3, the h/t_f is the same as *D*/*t* in BS 449, and the slenderness is identical. The main body of the table gives the proportion of the yield strength to be used when calculating the buckling resistance. There is some conservatism, since tables like this assume the most onerous value of C_1, but the result is a very simple table.

In practice, most designers use member resistance tables or software – which will deal with any (apparent) complexity with ease.

Buckling resistances – the outcomes

The theory is interesting, but what is the result? For flexural buckling, the calculated resistances are almost identical. For lateral torsional buckling, the resistance calculated to the Eurocode can be considerably higher than that according to BS 5950 – some 25% increase for a 7 m 533 × 210 UKB. This increased resistance may not be significant if deflection or other SLS criteria govern, but it is a significant advantage when the buckling resistance is the governing check.

Combined bending and axial load

The expression to verify in-plane and out-of-plane buckling are not for the faint-hearted. They are equivalent to the 'more exact' approaches in BS 5950,

Table 3.3. Presentation of lateral torsional buckling tables ( )

$\lambda_z = L/i_z$	$\bar{\lambda}_z$	h/t_f									
		5	10	15	20	25	30	35	40	45	50
30	0.35	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
40	0.46	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.99	0.99	0.99
50	0.58	1.00	0.99	0.96	0.95	0.95	0.94	0.94	0.94	0.94	0.94
60	0.69	1.00	0.95	0.92	0.91	0.90	0.89	0.89	0.88	0.88	0.88
70	0.81	1.00	0.93	0.88	0.86	0.85	0.84	0.83	0.83	0.82	0.82
80	0.92	0.98	0.90	0.85	0.82	0.80	0.78	0.77	0.77	0.76	0.76
90	1.04	0.97	0.87	0.81	0.77	0.75	0.73	0.72	0.71	0.71	0.70
100	1.15	0.95	0.85	0.78	0.73	0.70	0.68	0.67	0.66	0.65	0.64

and on inspection designers will see the same fundamentals in both standards. Unfortunately, the Eurocodes in general are not noted for 'simple' alternatives – the expressions for combined bending and axial load are one area where simple expressions would be welcome, at least for manual design. In UK practice, the most common occurrence of combined bending and axial load is when designing columns in simple construction. In this situation, http://www.access-steel.com provides NCCI that makes the verification straightforward, as the interaction expression becomes:

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{min,b,Rd}}} + \frac{M_{\mathrm{y,Ed}}}{M_{\mathrm{b,Rd}}} + 1.5\frac{M_{\mathrm{z,Ed}}}{M_{\mathrm{cb,z,Rd}}} \leq 1$$

In most orthodox cases, the moments are either zero (no net moment) or small and the column verification is dominated by the axial load. Software will be used by most designers using the general expressions in the Eurocode, and there are a number of design aids available from the SCI at www.steelbiz.org.

Connections

There are no significant changes in connection component strengths designed according to BS EN 1991-1-8. Bolts are as strong as they always have been (92 kN for an M20 8.8 conforming to BS 5950 becomes 94 kN in the Eurocode). Welds also have about the same resistance. There are dramatic increases in the calculated bearing resistance. BS 5950 provided a bearing strength that was chosen to limit the deformation at working load to 1.5 mm. The Eurocode has no such restriction, and therefore the resistances are much higher. The UK national annex notes that there may be some circumstances where reduced deformation is desirable and provides an alternative γ factor – bringing the bearing resistance close to the BS 5950 value.

In a very significant departure for UK practice, the Eurocode requires that connections be classified – meaning that for example, designers should demonstrate that their connection detail really is nominally pinned, if that has been assumed. The Eurocode reflects practice in some other European countries, where connection stiffness is calculated. Thankfully, there is some relief in that the Eurocode states that connections may be classified on the basis of previous satisfactory performance as an alternative to calculation. UK designers will be relieved that the UK national annex goes further and states that connections designed in accordance with the 'Green Books' on simple and moment connections can be considered as simple and rigid respectively.

Caution is urged for connections outside the familiar details in those books, as a demonstration of connection stiffness by calculation may be required. This is not a trivial calculation and usually demonstrates that a simple connection is not as 'pinned' as was hoped.

The UK methods of designing simple connections have been largely followed in European guides, which calculate the resistance of each component and identify the minimum resistance. An emerging issue is that tying requirements depend on building classification (this is covered in the UK Building Regulations and BS EN 1991-1-7) and this in turn means that some relatively low storey structures must accommodate relatively high tying forces through the beam to column connection. The much-loved partial depth flexible end plate has a relatively low resistance in tying, leading to the use of full depth end plates or thicker end plates. Additional standard details have been prepared which comprise slightly thicker end plates, welded to both flanges. The (enhanced) resistance of these new details have been calculated in accordance with BS EN 1993-1-1, and will be published during 2010.

Support tools

A wealth of support is already available, including NCCI, design guides, worked examples books of member resistance (the 'Blue' and 'Red' books) and concise guides. A number of publishers have guidance available. As might be expected, the Steel Construction Institute has published a range of guides, with support from BCSA and Tata. The 'Concise Guide' deserves a special mention as it tries to offer guidance on the common cases of design, from loading through to the detailed design checks. More guidance, covering composite members, simple connections, bridge design, fire engineering and other topics will be published in 2010 – most orthodox issues will be covered. The steel sector maintains the NCCI[1)] website (http://www.steel-ncci.co.uk) for steel-related information. Whilst this material is likely to be technically

[1)] Whilst this material is likely to be technically authoritative, not all of it has been reviewed by the UK national committee and users should satisfy themselves of its fitness for their particular purpose. In particular, they should be aware that material indicated as not having been endorsed by the committee might contain elements that are in conflict with the Eurocode. (Source National Annex to EN 1993-1-1:2005) authoritative, not all of it has been reviewed by the UK national committee, and users should satisfy themselves of its fitness for their particular purpose. In particular, they should be aware that material indicated as not having been endorsed by the committee might contain elements that are in conflict with the Eurocode.

As the Eurocodes are used, more NCCI will be required and will be added to this website.

Conclusions

This chapter has introduced some of the technical issues surrounding the design of steel structures. The author's conclusion is that there are no significant technical challenges – once designers have amassed modest experience they will appreciate that the design processes are straightforward and familiar, albeit dressed in slightly different wrapping to existing standards. There will be issues of familiarity ('which Eurocode Part, and which clause?') but these will diminish with time. Dealing with the sheer numbers of documents, each with a national annex, and the management of the change, especially across several materials, is likely to be the larger challenge.

The second conclusion is that the influence of the national annex should not be underestimated. The examples referred to in this chapter are not meant to be exhaustive or to imply that they are the most significant – they are simply examples to demonstrate the importance of the national annex. A careful review of each national annex and its impact is strongly recommended.

The third conclusion is that there are some economic advantages in designing in accordance with the Eurocodes (based on the technical changes alone). Loads are reduced, and resistance is increased in some areas. For multi-storey frames the reduction in loads will be a significant saving. For lighter structures significantly affected by wind actions, such as portal frames, the jury is still out.

The final observation is that designers already have a great deal of support available, which will ease the transition. Much of this support is online, and entirely free.

Eurocode 4: Design of composite steel and concrete structures

Dr Stephen Hicks, Manager Structural Systems, Heavy Engineering Research Association, New Zealand

Introduction

BS EN 1994 (Eurocode 4) is the Structural Eurocode that deals with composite steel and concrete structures. It replaces the following national standards: BS 5400-5, BS 5950-3.1 and BS 5950-4. Eurocode 4 consists of three Parts:

- Part 1-1, *General rules and rules for buildings* (BS EN 1994-1-1);
- Part 1-2, *General rules — Structural fire design* (BS EN 1994-1-2); and
- Part 2*, General rules and rules for bridges* (BS EN 1994-2).

To enable Eurocode 4 to be used, designers also need to make reference to the national annex, which includes the national decision for Nationally Determined Parameters (NDPs), the national decision regarding the use of informative annexes and reference to Non-Conflicting Complementary Information (NCCI). For BS EN 1994-1-1 and BS EN 1994-1-2, the website http://www.steel-ncci.co.uk will provide all the necessary NCCI, whilst for BS EN 1994-2 NCCI is given in PD 6696-2. In the interests of improving free circulation of products and services in Europe, it is intended to reduce the number of NDPs in the future, thereby leading to a gradual alignment of safety levels across the member states. As a first step in this process, the European Commission Joint Research Centre (JRC) has commenced a pilot project that is considering the harmonization of NDPs, whose initial focus is Eurocode 2, Eurocode 3 and Eurocode 4 (http://eurocodes.jrc.ec.europa.eu/showpage.php?id=52).

To assist designers in understanding Eurocode 4, references [1], [2] and [3] provide background information on the origin and objectives of the code provisions, which are supplemented by a selection of worked examples that illustrate the use of a particular clause. In addition, background information is freely available through the Eurocodes website of the JRC (http://eurocodes.jrc.ec.europa.eu/).

The objective of this chapter is to provide an overview of the key aspects to Eurocode 4 and consider the principal changes for UK designers. The convention used is that, when the provisions are similar in different parts of this Structural Eurocode, Eurocode 4 is referenced. However, when the rules are specific to a certain type of structure, the relevant part is identified (e.g. BS EN 1994-1-1 for buildings).

Materials

Structural steel

Although the use of structural steel with a nominal yield strength of not more than 460 N/mm^2 is permitted in bridge designs conforming to BS 5400-5, Eurocode 4 offers opportunities for building designers, where previously a yield strength of not greater than 355 N/mm^2 was allowed in BS 5950-3.1. According to Eurocode 3, the modulus of elasticity for steel should be taken as 210 kN/mm^2, rather than the value of 205 kN/mm^2 given in BS 5400 and BS 5950.

Concrete

The strength and deformation characteristics for normal weight and lightweight concrete are given in Eurocode 2. The compressive concrete strengths used in the design rules in according to Eurocode 4 are based on cylinder strengths. Strength classes are defined as C*x*/*y* for normal weight concrete and LC*x*/*y* for lightweight concrete, where *x* and *y* are the characteristic cylinder and cube compressive strengths respectively. For example, C25/30 denotes a normal weight concrete with a characteristic cylinder strength of 25 N/mm^2 and a corresponding cube strength of 30 N/mm^2.

While BS 5950-3.1 covers the use of concrete grades C25/30 to C40/50 and LC20/25 to LC32/40, the range of concrete grades that are permitted in designs conforming to Eurocode 4 are much wider at C20/25 to C60/75 and LC20/22 to LC60/66 respectively. Although Eurocode 2 provides guidance for lightweight concrete with dry densities of between 800 kg/m^2 and 2000 kg/m^2, it is unlikely that a density of less than 1750 kg/m^3 will be used in composite design, owing to the fact that this is the lowest value that is permitted in the Eurocode 4 equations for evaluating the resistance of headed stud connectors.

Profiled steel sheeting

Yield strengths of 280 N/mm^2 and 350 N/mm^2 are the common grades for steel strip in the UK. Typically, profiled steel sheeting (or decking) is galvanized for durability purposes and, for internal environments, a total zinc coating of 275 g/m^2 is normal. Grades of steel for profiled steel sheeting are specified in BS EN 10326 (this replaces BS EN 10147, which is the reference given in the current version of BS EN 1994-1-1), which distinguishes both the yield strength and the level of zinc coating. For example, the designation S 280 GD + Z 275 means 280 N/mm^2 yield strength and 275 g/m^2 of zinc coating.

The rules in BS EN 1994-1-1 are only appropriate for profiled steel sheeting thicknesses above a certain bare metal thickness. The UK national annex uses the recommended value of $t \geq 0.70$ mm. Although an identical minimum sheet thickness is given in BS 5950-4, bare metal thicknesses of between 0.86 mm to 1.16 mm have generally been used in the UK to date. The thickness of a 275 g/m^2 zinc coating is equivalent to approximately 0.02 mm on each face, resulting in overall sheet thicknesses commonly used in the UK of between 0.9 mm to 1.2 mm. For design calculations the smaller bare metal thickness should be used.

Reinforcement

In a similar way as BS 5950-3.1, to simplify calculations the modulus of elasticity of the reinforcement may be taken as equal to the value for structural steel in Eurocode 4 (i.e. 210 kN/mm^2 rather than 200 kN/mm^2 given in Eurocode 2).

Shear connectors

Headed stud connectors should be supplied according to BS EN ISO 13918 (rather than EN 13918, which is the reference incorrectly given in Eurocode 4).

To distinguish studs used for shear connectors, the designation SD is used, for example, SD 19 × 100, is a headed stud shear connector with a 19 mm diameter shank and a nominal height of 100 mm. Due to the limitations to the Eurocode 4 design equations for calculating the resistance of headed stud connectors, the stud shank diameters that will be used in practice are likely to be between 16 mm and 25 mm for solid concrete slabs, and not greater than 19 mm for studs through-deck welded within the ribs of profiled steel sheeting. The performance of other types of shear connector may be evaluated from standard tests given in the informative Annex B.2 of BS EN 1994-1-1, in the absence of guidelines for a European Technical Approval (ETA).

In Eurocode 4, the nominal height of the stud rather than the length-after-welding (LAW) is used in the design equations. However, LAW is needed for detailing purposes, and is sometimes used to ensure that limits to design rules are satisfied (e.g. LAW is required to determine whether a stud may be taken as ductile in the rules for partial shear connection). As a consequence of this, two values of stud height need to be considered by the designer: the nominal height for calculating resistance; and LAW when detailing the shear connection. Traditionally, the LAW is taken as 5 mm shorter than the nominal height.

Composite beams

Effective width of concrete flanges to composite beams for shear lag

The rules for the effective width in Eurocode 4 are simpler than BS 5400-5, but similar to those in BS 5950-3.1. The effective width at the ultimate limit state is taken as a constant value for the middle portion of the span and tapers towards the points of zero moment, as shown in Figure 4.1 (as opposed to BS 5950-3.1 where a constant width is taken along the full length for simply-supported beams); similar results for effective widths of steel plated structural elements can be calculated from BS EN 1993-1-5. In addition, when multiple shear connectors are provided, the effective width may be increased by the distance between the outermost shear connectors measured from their centre-lines, b_0 (see Figure 4.1). However, for the serviceability limit state, the Eurocode 4 provisions are similar to BS 5950-3.1 in that a constant effective breadth may be assumed to act over the entire span, based on the mid-span value.

Key

1 $L_e = 0.85L_1$ for $b_{eff,1}$
2 $L_e = 0.25(L_1 + L_2)$ for $b_{eff,2}$
3 $L_e = 0.70L_2$ for $b_{eff,1}$
4 $L_e = 2L_3$ for $b_{eff,2}$

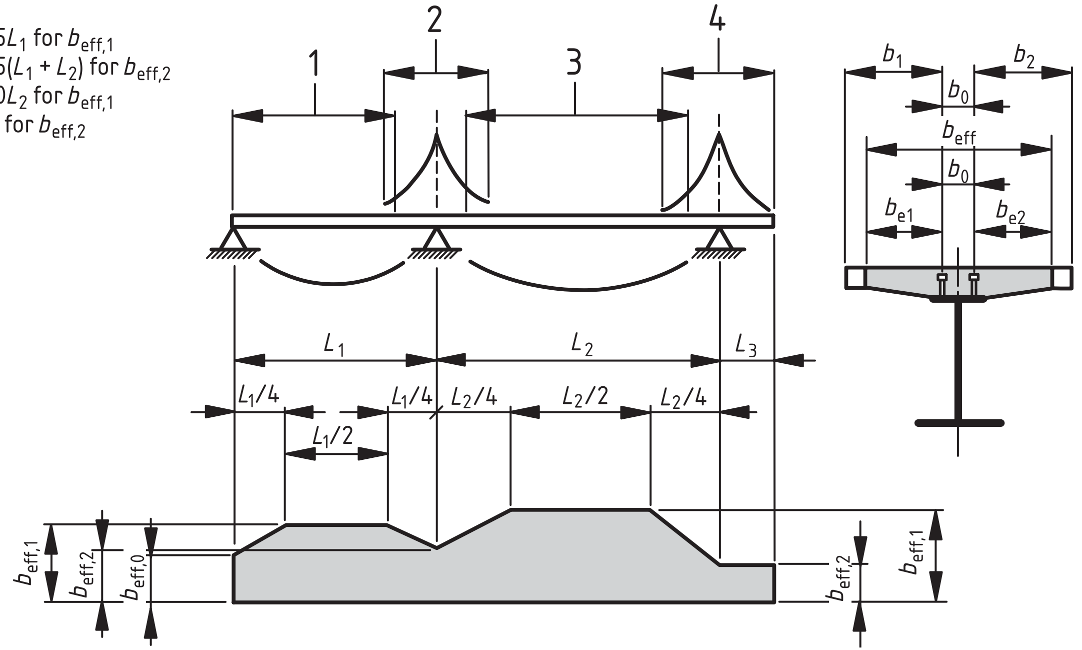


Figure 4.1. Equivalent spans, for effective width of concrete flange

In both BS 5950-3.1 and Eurocode 4, the maximum value of the effective width $b_{e1} = b_{e2}$ = span/8 on each side of the beam (see Figure 4.1). As well as considering this limit, the width assumed in design must not exceed the actual slab width available, which is particularly relevant to edge beams and beams adjacent to openings. The rules in Eurocode 4 are more generous for cases when the slab is spanning parallel to the span of the beam in that, in BS 5950-3.1, the width assumed in design could not exceed 80% of the actual slab width available.

Creep and shrinkage

One of the differences from previous UK practice is that the elastic modulus for concrete under short-term loading is a function of its grade and density. As a consequence of this, instead of the short-term value, n_0, of 6 and 10 for normal weight and lightweight concrete respectively, a range of values should be used. For design conforming to Eurocode 4, n_0 ranges between: 5.2 to 6.8 for normal concrete; and 8.3 to 10.8 for lightweight concrete with a dry density $\rho = 1750\,\text{kg/m}^3$.

In BS 5950-3.1, the effective modular ratio that should be used in design is based on a consideration of the short- and long-term modular ratio, and the proportion of the total loading that is long term. However, BS EN 1994-1-1 introduces a useful simplification for composite beams in buildings in that the modular ratio may be taken as $2n_0$ for both short- and long-term loading if:

- first-order global analysis is acceptable (which is expected to occur in the majority of cases);
- the floor is not mainly intended for storage; and
- the floor is not prestressed by controlled imposed deformations.

Shear connection

Partial shear connection

Ductile shear connectors are defined as those having sufficient deformation capacity to justify the assumption of ideal plastic behaviour of the shear connection (measured in terms of the slip at the interface between the steel beam and the concrete slab). Sufficient slip capacity enables the longitudinal shear to be redistributed between the shear connectors before any of them fail, such that they may be taken to be equally loaded at the ultimate limit state. In these situations, it is permitted to space the connectors uniformly, which is helpful when the connectors are used with profiled steel sheeting, due to the fixed pitch of the ribs. Unlike BS 5950-3.1, whose only requirement is that other types of shear connectors should have at least the same deformation capacity as headed studs, Eurocode 4 specifies that a shear connector may be taken to be ductile if its characteristic slip capacity, δ_{uk}, is at least 6 mm. In situations when the shear connector is not a headed stud, δ_{uk} may be evaluated from the standard test given in Annex B.2 of BS EN 1994-1-1.

According to BS EN 1994-1-1, headed studs with a shank diameter, d, of between 16 mm and 25 mm, and an overall length after welding (LAW) of not less than $4d$ may be considered ductile within defined limits to the degree of shear connection, η. Unlike BS 5950-3.1, where the limits to the degree of shear connection depended only on the beam span, the limits in BS EN 1994-1-1 are a function of the beam span, the steel grade and whether the steel section is symmetric or asymmetric (defined by the ratio of the bottom flange area to top flange area of the steel section). The maximum asymmetry that is permitted is for steel sections with a bottom flange area equal to three times the area of the top flange. For steel sections in which the ratio of flange areas is between 1 and 3, linear interpolation is permitted. A graphical representation of the degree of shear connection requirements in BS 5950-3.1 compared with BS EN 1994-1-1 is presented in Figure 4.2. As can be seen from Figure 4.2, for symmetric steel sections, a much lower degree of shear connection is permitted than in BS 5950-3.1.

A third set of rules where headed stud connectors may be considered as ductile over a wider range of spans is given in BS EN 1994-1-1. However, these are more restrictive in scope and only apply to profiled steel sheeting spanning perpendicular to the supporting beam, with ribs not greater than 60 mm in height and one 19 mm diameter stud per rib. Moreover, this third set of rules can only be used when the simplified method is used (where the composite moment resistance is linearly interpolated between full shear connection and no shear connection), as opposed to the rules in Figure 4.2 where the traditional stress-block method is used, which gives a larger lever arm and moment resistance.

The use of non-ductile shear connectors is permitted in Eurocode 4 (such as headed studs used outside the ranges given in Figure 4.2, or block connectors).

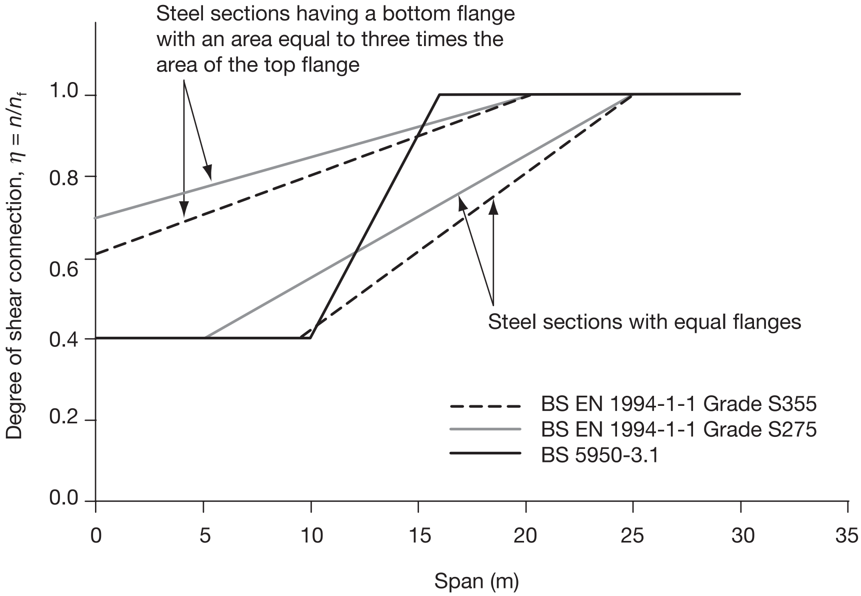


Figure 4.2. Minimum shear connection requirements from BS 5950-3.1 and BS EN 1994-1-1

However, the spacing of the shear connectors must be based on an elastic analysis of the longitudinal shear.

Resistance of shear connectors embedded in solid slabs and concrete encasement

Although only design rules for headed stud connectors are given in Eurocode 4, the UK national annex to BS EN 1994-2 provides guidance for block connectors with hoops through PD 6696-2. Specific design rules for horizontally lying studs are provided in Annex C of BS EN 1994-2 which, according to the UK national annex to BS EN 1994-1-1, may also be used for buildings.

Unlike BS 5950-3.1 and BS 5400-5, where the characteristic resistances of headed stud connectors were presented in tabular form, the stud resistance in Eurocode 4 is taken to be the lesser of two equations (one representing stud shank failure, the other representing crushing of the concrete around the stud). A comparison of the characteristic resistances of typical 19 mm

Table 4.1. Characteristic resistances of 19 mm diameter × 95 mm LAW stud connectors embedded in normal weight concrete

Standard	Characteristic resistances of shear connectors (kN) for concrete grade				
	C20/25	C25/30	C30/37	C35/45	C40/50
Eurocode 4	81	93	104	113	113
BS 5400 and BS 5950-3.1	95	100	106	114	119

Table 4.2. Characteristic resistances of 19 mm diameter × 95 mm LAW stud connectors embedded in lightweight concrete (with a dry density ρ = 1750 kg/m³)

Standard	Characteristic resistances of shear connectors (kN) for concrete grade				
	LC20/22	LC25/28	LC30/33	LC35/38	LC40/44
Eurocode 4	64	74	83	91	99
BS 5400 and BS 5950-3.1	83	88	92	97	102

diameter studs embedded in solid concrete slabs is presented in Table 4.1 and Table 4.2 for normal weight and lightweight concrete respectively.

Unlike BS 5950-3.1, where the design stud resistance is reduced in hogging moment regions, in Eurocode 4 it is assumed that the design resistance is not dependent on whether the surrounding concrete is in compression or tension. Although test evidence suggests this assumption is slightly unconservative for hogging moment regions [4], this is compensated by the fact that only full shear connection is permitted by BS EN 1994-1-1 in these areas.

While BS 5950-3.1 and BS 5400-5 recognize that appropriate resistance to uplift should be provided by the shear connectors, only BS 5400-5 provides specific rules on the influence of tension on the shear resistance of headed studs. According to Eurocode 4, the design shear resistance of headed studs, P_{Rd}, may be assumed to be unaffected, provided that the design tensile force does not exceed $0.1P_{Rd}$; for situations when the design tensile force exceeds this value, the connection is not within the scope of Eurocode 4. However, for situations where significant tension forces may develop in shear studs (such as may be encountered over long web-openings, tension-field action, etc.), guidance to UK designers is given in PD 6696-2.

Design resistance of headed studs used with profiled steel sheeting in buildings

The BS EN 1994-1-1 reduction factors that are applied to stud connectors welded within the ribs of profiled steel sheeting are calculated using identical equations to those in BS 5950-3.1, except that a lower multiplier is used for cases when the sheeting ribs are perpendicular to the supporting beams. Also, while the limiting values to the reduction factors in BS 5950-3.1 were based on the number of studs per rib, the limits in BS EN 1994-1-1 are a function of the number of studs per rib, the thickness of the sheet and whether the studs are through-deck welded or welded through holes in the sheet. Unlike BS 5950-3.1, no reduction factor equations are provided for more than two studs per rib.

The geometry of existing UK profiled steel sheets have been designed such that the limiting value dominates, so the reduction factors in BS EN 1994-1-1 are independent of the geometry and are therefore based on the number of studs per rib and the orientation of the sheet. As a consequence of this, for through-deck welded 19 mm diameter × 95 mm LAW studs, the reduction factor values from BS EN 1994-1-1 are identical to those given in BS 5950-3.1 for sheet thicknesses greater than 1.0 mm, but up to 15% lower for sheet thicknesses less than 1.0 mm. Nevertheless, when concrete grades less than C35/45 and LC40/44 are used, the resistance of headed stud connectors will be lower than those given by BS 5950-3.1, irrespective of the sheet thickness (see Table 4.1 and 4.2).

Detailing of the shear connection

One of the significant differences in the detailing rules to Eurocode 4 compared to BS 5950-3.1 is the requirement that the underside of the head of a stud should extend not less than 30 mm clear above the bottom reinforcement to provide adequate resistance to separation; this rule appears to have been developed from a consideration of the performance of studs in solid slabs, or composite slabs with shallow re-entrant profiled steel sheeting. In 60 mm deep profiled steel sheets commonly used in the UK, the presence of a shallow re-entrant stiffener to the top flange of the sheet results in an overall depth closer to 70 mm, meaning that this detailing rule cannot be achieved for typical 19 mm diameter × 95 mm LAW studs. Nevertheless, recent full-scale beam tests have indicated that this rule could be relaxed for typical 60 mm deep profiled steel sheets used in the UK [5].

Design resistance to longitudinal shear in concrete slabs

In evaluating the amount of transverse reinforcement required to prevent longitudinal splitting caused by the forces from the shear connectors, Eurocode 4 refers to the provisions in Eurocode 2 for reinforced concrete T-beams. The rules in Eurocode 2 are based on a truss analogy, where it assumed that successive concrete struts form in the flange to the beam with the transverse reinforcement acting as ties to maintain equilibrium and prevent the concrete struts from rotating (see Figure 4.3). This approach is a significant departure to the rules for transverse reinforcement in BS 5400-5 and BS 5950-3.1, which were developed from a semi-empirical relationship.

Like BS 5950-3.1 and BS 5400-5, the design longitudinal shear resistance of the concrete slab should exceed the design resistance of the shear connectors to ensure that the more ductile shear connectors are the critical design case. Where a combination of precast and in-situ concrete is used, the longitudinal shear resistance should again be evaluated according to Eurocode 2, but in these situations using the provisions for shear at the interface for concrete

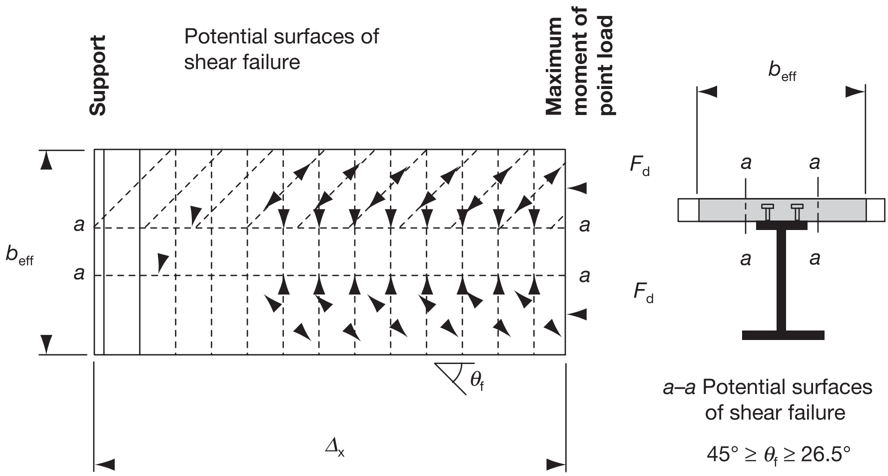


Figure 4.3. Truss model for transverse reinforcement

cast at different times. These rules are different to those currently recommended in UK practice [6].

In a similar way as in BS 5950-3.1, when profiled steel sheeting spans perpendicular to the supporting beam and is either continuous, or discontinuous but anchored (from the provision of through-deck welded stud connectors), the sheet may be taken to contribute to the transverse reinforcement. However, for the case when the sheets are discontinuous and anchored, the rules in BS EN 1994-1-1 are more consistent than BS 5950-3.1 and BS 5950-4, in that the basis for calculating the bearing resistance of the stud is identical for both transverse reinforcement considerations and end anchorage in composite slabs.

Serviceability limit state

Deflections

The additional deflection due to partial shear connection need not be considered if the shear connection is:

- designed according to the methods for headed studs in BS EN 1994-1-1 (see Figure 4.2);
- the degree of shear connection, η, is not less than 50%; and
- when the ribs of the profiled steel sheet are perpendicular to the supporting beam their height does not exceed 80 mm.

Shrinkage of the concrete results in forces on the shear connectors to act in the opposite direction to that due to the vertical loads, and can therefore be neglected when designing the shear connection. However, the shrinkage forces can cause the beam to deflect in the same way as if the beam was subject to vertical loading, which leads to additional deflections and flexural stresses. In BS 5950-3.1, it was not necessary to consider the effects of shrinkage if the calculation procedures provided in that Standard were adopted. According to BS EN 1994-1-1, the additional deflection due to shrinkage need not be included in design if the span-to-depth ratio of the beam is not less than 20 and normal weight concrete is used. For other cases, guidance is given by Johnson and Anderson [1].

Irreversible deformation

As opposed to BS 5950-3.1, there are no specific requirements to limit stresses at the serviceability limit state in BS EN 1994-1-1. However, to ensure that it is appropriate to base the calculations for deflections on elastic theory, it is considered good practice to use similar limitations as BS 5950-3.1. On this basis, it is recommended [7] that in designs conforming to BS EN 1994-1-1 the calculated stresses should be limited to the yield strength of the steel, f_y, and the concrete stress to $0.63f_{ck}$.

Vibrations

Owing to the fact that limits to vibrations are material-independent, Eurocode 4 refers designers to BS EN 1990. For vibration limits in buildings, BS EN 1990, Annex A1.4.4 refers to ISO 10137. However, no guidance is given to the designer on how these limits should be verified; it is expected that, for steel-framed buildings, an appropriate NCCI will be given, such as reference [8]. For bridges, specific vibration limits are provided in Annex A2.4 of BS EN 1990.

Crack widths

Where composite beams and composite slabs are designed as simply-supported, but the slab is continuous, a minimum percentage of reinforcement should be provided over the intermediate supports. According to BS 5950-4, reinforcement equivalent to 0.1% of the cross-sectional area of the concrete should be provided as a minimum for unpropped construction. However, UK industry has already moved away from this value and adopted the following BS EN 1994-1 provisions as good practice when the control of crack widths is not required:

- 0.2% of the cross-sectional area of the concrete (taken as the depth above the ribs of the sheeting, h_c, for composite slabs) for unpropped construction;
- 0.4% of the cross-sectional area of the concrete (taken as the depth above the ribs of the sheeting, h_c, for composite slabs) for propped construction.

When limits to the crack widths are required, reference should be made to Eurocode 2 for composite slabs and slabs to beams.

Design for fire resistance

The fire resistance of a composite beam may be evaluated using the bending moment resistance model in BS EN 1994-1-2, which is similar to the moment capacity method given in BS 5950-8. When the ribs of the profiled steel sheeting are perpendicular to the supporting beam, voids are created between the sheeting and the top flange of the steel beam. Unlike BS 5950-8, where limiting temperatures were only provided when the voids were filled with non-combustible filler, according to BS EN 1994-1-2 the voids may be ignored if at least 85% of the surface of the top flange is in contact with the slab. As a consequence of this, the voids do not need to be filled for re-entrant profiles, but they must be filled for trapezoidal profiles (or the effect of the voids on the beam temperature must be considered).

An alternative method for evaluating the fire resistance of a composite beam is the critical temperature model in BS EN 1994-1-2, which is used to estimate the critical temperature of the lower flange of the steel beam under a given sagging bending moment. Although this method is simple, for a composite beam designed for partial shear connection at ambient temperature, the critical temperature method is likely to be more conservative compared to that achieved using BS 5950-8.

Composite columns

Rules for composite columns in buildings were intended to be provided in BS 5950-3.2, but this standard was never published. However, rules for composite columns were published in BS 5400-5, and have been used in the UK for the design of bridge piers. The rules for composite columns in Eurocode 4 are appropriate for concrete filled steel hollow sections, fully concrete-encased and partially concrete-encased steel H-sections. The advantages of using composite columns are that they possess a high bearing resistance and, in buildings, significant periods of fire resistance can be achieved without the need for applied external protection.

Composite joints

Although design guidance for composite beam-to-column connections has been available since 1998 [9], the design rules are formalized through the publication of BS EN 1994-1-1. The benefit of using composite connections in braced frames is that beam depths and section sizes can be reduced, improved serviceability performance is achieved (in terms of deflections) and, due to the improved continuity between the frame members, greater robustness is possible.

Composite slabs

Flexure

The *m-k* method in BS 5950-4 is the traditional approach for evaluating the longitudinal shear resistance of composite slabs; however, this method has limitations and is not particularly suitable for the analysis of concentrated line and point loads. As well as the *m-k* method, in BS EN 1994-1-1 another approach known as the partial connection method is given, which is based on the principles of partial shear connection. This method provides a more logical approach to determine the slab's resistance from applied concentrated line or point loadings, but may only be used when ductile longitudinal shear behaviour has been demonstrated by tests on composite slabs.

Both the *m-k* and partial connection method in BS EN 1994-1-1 rely on tests on composite slabs to evaluate the longitudinal shear strength, or 'shear bond' value, for the variables under investigation. However, design values that have been evaluated from tests according to BS 5950-4 cannot be used directly in Eurocode 4, unless they have been converted by a method such as that described in [10]. It is expected that, once the national standards are withdrawn, design tables and software according to the Eurocodes will be provided by profiled steel sheeting manufacturers for their specific products.

Concentrated point and line loads

Concentrated point and line loads often occur in buildings from, for example, temporary props during construction, wheel loads, columns, solid masonry partitions, etc. In these situations, the effect of the smaller effective slab width available for bending and vertical shear resistance needs to checked at the locations of these loads. The BS EN 1994-1-1 equations for determining the effective width of composite slabs are identical to those given in BS 5950-4, with the exception that their applicability is limited to cases when the ratio of the sheet height to the overall slab depth h_p/h does not exceed 0.6. Moreover, although an identical nominal transverse reinforcement area of not less than 0.2% of the area of concrete above the ribs of the sheet is specified in BS EN 1994-1-1, a significant difference is that this level of reinforcement is only appropriate for characteristic imposed loads not exceeding 7.5 kN for concentrated loads, and 5.0 kN/m^2 for distributed loads. In situations when this loading is exceeded, the appropriate transverse reinforcement should be determined in accordance with Eurocode 2.

Vertical shear

The vertical shear resistance of a composite slab should be determined using Eurocode 2, which depends on the effective depth of the cross-section to the centroid of the tensile reinforcement. Although not specified in BS EN 1994-1-1, in BS 5950-4 and the ENV version of BS EN 1994-1-1 it was permitted to take the profiled steel sheeting as the tensile reinforcement provided that it was fully anchored beyond the section considered. However, for heavily loaded slabs additional reinforcement may be required at the support when the profiled steel sheeting is discontinuous and only has limited anchorage.

Design for fire resistance

The required fire performance of floor slabs is defined by the Approved Document B to the UK National Building Regulations. The Approved Document requires the slab performance to be assessed based on criteria for insulation (criterion I), integrity (criterion E) and load bearing capacity (criterion R). In BS EN 1994-1-2, it may be assumed that composite slabs satisfy the integrity criterion. Moreover, according to BS EN 1994-1-2, composite slabs that have been designed to BS EN 1994-1-1 may be assumed to possess 30 min fire resistance when assessed according to the load bearing capacity criterion. Nevertheless, the slab's ability of achieving the insulating criterion still needs to be verified.

The insulation criterion is satisfied by providing adequate slab thickness to ensure that the temperature of the unexposed surface of the slab does not exceed 140 °C. The UK national annex to BS EN 1994-1-2 provides a table of recommended slab thicknesses for both trapezoidal and re-entrant profiles to satisfy the insulation requirements for common periods of fire resistance. These slab thicknesses are identical to those given in BS 5950-8.

Despite the fact that Annex D of BS EN 1994-1-2 provides a calculation model for estimating the fire resistance of composite slabs, the UK national annex does not recommend its use, owing to the fact that many UK profiled steel sheets are outside the limits to its field of application. In an attempt to resolve this issue, alternative design temperatures based on BS 5950-8 are presented in the UK national annex.

Typically, design tables that satisfy the load bearing criterion are given by profiled steel sheeting manufacturers, which are based on the extended application of a single fire test on a particular product. Although the extended application of fire test results in the UK is already based on a design model that is in the spirit of BS EN 1994-1-2, extending the application of fire tests will be formalized in the future through the publication of a series of European Standards with the designation EN 15080. For projects in other European countries, where the use of Annex D of BS EN 1994-1-2 is recommended, it is likely that the manufacturer's fire design tables will be the only valid method of design for UK profiles; in particular, when the contribution of the tensile resistance of the profiled steel sheet is included in the calculation of the sagging moment resistance (a practice that has hitherto been included in UK design, which often eliminates the need for reinforcement bars within the ribs).

Conclusions

Eurocode 4 brings both benefits and challenges to UK designers who are familiar with the earlier national standards for composite steel and concrete structures. To assist designers in the transition to the Eurocodes, the Steel Construction Institute (SCI) have issued a suite of design guides that provide advice on designing structural elements and frames.. In addition to the design guides, the European steel industry's multilingual Eurocode 3 and Eurocode 4 website, Access Steel (www.access-steel.com), contains further guidance.

References

[1] Johnson R.P. and Anderson D. *Designers' Guide to EN 1994-1-1: Eurocode 4: Design of Composite Steel and Concrete Structures, Part 1-1: General Rules and Rules for Buildings*, Thomas Telford, London, 2004

[2] Moore D., Bailey C., Lennon T. and Wang, Y. *Designers' Guide to EN 1991-1-2, EN 1992-1-2, EN 1993-1-2 and EN 1994-1-2*, Thomas Telford, London, 2007

[3] Hendy C.R. and Johnson R.P. *Designers' Guide to EN 1994-2 Eurocode 4: Design of composite steel and concrete structures, Part 2: General rules and rules for bridges*, Thomas Telford, London, 2006

[4] Johnson R.P., Greenwood R.D. and van Dalen K. Stud shear-connectors in hogging moment regions of composite beams, *The Structural Engineer*, Vol. 47, No. 9, September 1969, pp345–350

[5] Hicks S.J. Strength and ductility of headed stud connectors welded in modern profiled steel sheeting, *The Structural Engineer*, Vol. 85, No. 10, May 2007, pp32–38

[6] Hicks S.J. and Lawson R.M. *Design of Composite Beams using Precast Concrete Slabs*, SCI Publication 287, The Steel Construction Institute, Ascot, 2003, p92

[7] Rackham J.W., Couchman G.H., and Hicks S.J. *Composite Slabs and Beams using Steel Decking: Best Practice for Design and Construction* (Revised Edition), SCI Publication 300/MCRMA Technical Paper No. 13, The Metal Cladding and Roofing Manufacturers Association in partnership with the Steel Construction Institute, Wirral, 2009, p110

[8] Smith A.L., Hicks S.J. and Devine P.J. *Design of Floors for Vibration: A New Approach*, SCI Publication 354, Steel Construction Institute, Ascot, 2007, p124

[9] Couchman G.H. and Way A.G.J. *Joints in Steel Construction – Composite Connections*, SCI Publication 213, Steel Construction Institute, Ascot, 1998, p98

[10] Johnson R.P. *Models for the longitudinal shear resistance of composite slabs and the use of non-standard test data*, In Composite Construction in Steel and Concrete V, Leon R.T. and Lange J. (eds). ASCE, New York, 2006, pp157–165

Eurocode 5: Design of timber structures

Arnold Page, BSc, BD, MIWSc. Structural timber engineering consultant

Introduction

BS EN 1995 consists of three parts:

- Part 1-1: *General. Common rules and rules for buildings*
- Part 1-2: *General. Structural fire design*
- Part 2: *Bridges*

Each part has its own national annex. The three UK national annexes for Eurocode 5 contain only 10 substantive pages in total.

With BS EN 1990 and three standards which provide essential material properties (BS EN 338, *Structural timber – Strength classes*, BS EN 1194, *Timber structures – Glued laminated timber – Strength classes and determination of characteristic values*, and BS EN 12369, *Wood-based panels – Characteristic values for structural design*) Eurocode 5 replaces BS 5268-2, 3, 4 and 6.

Scope and contents

The three parts of Eurocode 5 provide the procedures and formulae required to determine the mechanical resistance, serviceability, fire resistance and some aspects of the durability of timber structures and their connections. Part 1-1, as its title states, provides general rules for the design of timber structures and of buildings in particular. Part 1-2 provides methods to ensure that specified periods of fire resistance can be achieved to avoid collapse and limit the spread of flame by maintaining the integrity of separating walls and floors. Part 2 supplements Part 1-1 by providing design methods related to the reliability of a whole bridge or major parts of it, whether it is made of timber or of timber acting in conjunction with concrete.

The materials covered comprise:

- structurally graded solid timber;
- glulam (glued laminated timber);
- LVL (laminated veneer lumber) and other structural reconstituted wood products;
- structurally certified wood-based panels – plywood, particleboard, OSB (oriented strandboard) and various types of fibreboard;
- metal fasteners and plates for connections;
- timber/concrete beam/decks in bridges.

Eurocode 5 covers the use of these materials per se as well as in components such as I-joists and in assemblies such as roof trusses, timber frame wall panels and portal frames.

Eurocode 5 does not provide:

- strength and stiffness properties for timber and wood-based materials;
- tables of pre-calculated values for the load-carrying capacity of metal fasteners
- information on the design of glued joints;
- design rules for earthquake resistance (which are given in BS EN 1998);
- non-structural aspects of design such as thermal and sound insulation.

Like the other material-specific Eurocodes, Eurocode 5 has to be used in conjunction with two further Eurocodes which apply to all structural materials: BS EN 1990 and BS EN 1991.

BS EN 1990 (Eurocode: *Basis of structural design*) explains how to calculate the design values of actions (usually loads) and material properties, and BS EN 1991 (Eurocode 1: *Actions on structures*) tabulates standard material weights and specifies imposed loads and wind loads, replacing BS 6399. Figure 5.1 illustrates the principal parts of the Eurocode system on which timber design depends.

Tables 5.1 to 5.3 summarize the contents of each of the three parts of Eurocode 5.

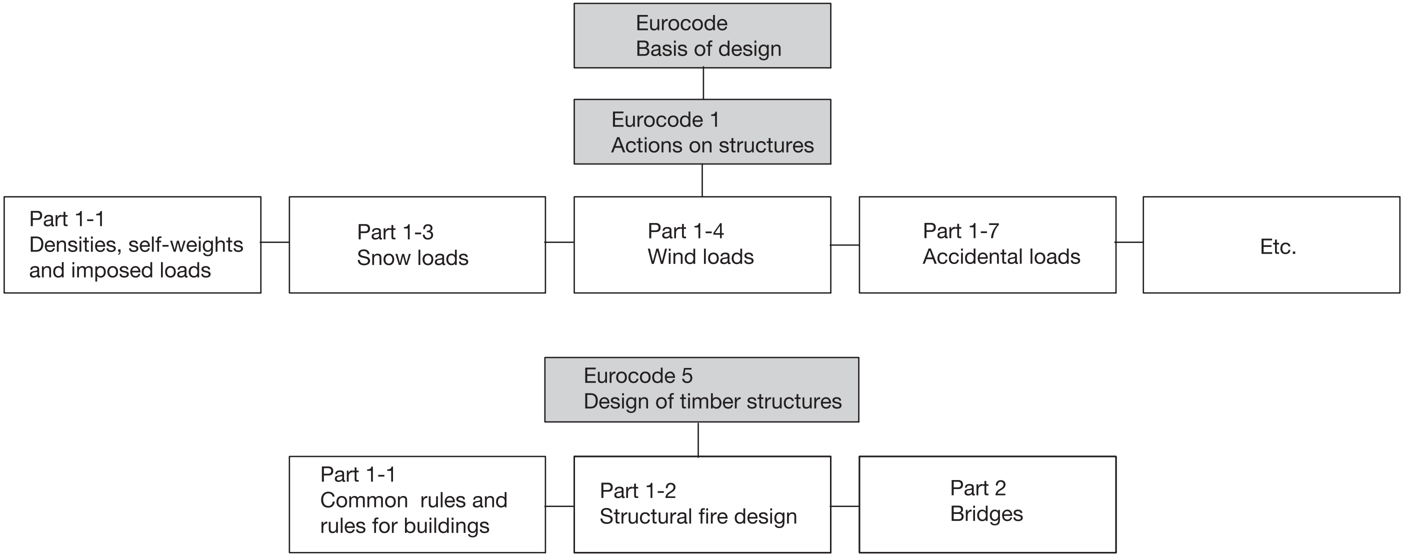


Figure 5.1. Eurocode system for structural timber design

Table 5.1. Contents of Part 1-1: *General – Common rules and rules for buildings*

	Section	Description
1	General	Scope, references, definitions and symbols
2	Basis of design	Supplements Eurocode 0 with timber-specific matters – service classes, load duration classes, creep and slip, plus partial safety factors for different timber based materials
3	Material properties	This section does not include properties as such, but gives tables of values to allow for the effects on timber materials of service class, load duration and creep
4	Durability	A page on durability which includes a useful table of specifications for the corrosion protection of metal fasteners
5	Basis of structural analysis	A short section covering mainly the analysis of assemblies, including frames (e.g. roof trusses) and arches (e.g. portal frames)
6	Ultimate limit states	An important section covering the design of beams, columns and ties, including tapered, curved and pitched cambered beams
7	Serviceability limit states	This explains how to calculate the initial and final slip in mechanically fastened joints, defines the components of deflection, and explains how to check the vibration of domestic floors. (The vibration performance of floors must be checked for floor design in Eurocode 5.) More precise recommendations for limiting the deflection of beams are given in the national annex, which also provides substantial additional guidance for the checking of floor vibrations
8	Connections with metal fasteners	The longest section in the code, Section 8 provides formulae for calculating the shear and withdrawal strength of mechanical fasteners and the strength of connections made with mechanical fasteners. It does not provide any tables of pre-calculated values. The fasteners covered are staples, nails, wood screws, bolts, steel dowels, punched metal plate fasteners (commonly called ‘nail plates’), split rings, shear plates and toothed-plate connectors

9	Components and assemblies	This section provides useful guidance on the design of items such as I-joists, stressed skin panels, horizontal diaphragms and bracing. It also provides two methods for designing wall panels to resist racking loads, the second of which is specified by the national annex for the design of timber framed walls in the UK
10	Structural detailing and control	Like its counterpart in BS 5268, this section is fairly short and relates mainly to connections
A	Block shear and plug shear failure at multiple dowel-type steel-to-timber connections	The first of three informative annexes, this one provides guidance on checking against the possibility that a complete block or plug of timber may shear out of a connection loaded in tension by multiple fasteners such as nails or bolts. The national annex restricts the use of this clause to specified minimum numbers of fasteners
B	Mechanically jointed beams	A design method for determining the strength and stiffness of nailed I-beams and box-beams which allows for slip at the connections. Although the national annex permits its use, the calculated bending stiffness does not always match experimental results
C	Built-up columns	Guidance for the design of built-up columns and trusses in which two or more parallel timber members are connected spaced apart by packing pieces. The trusses can be of 'V' or 'N' formation and the connections can be made with mechanical fasteners or be adhesively bonded
D	Bibliography	A minimal bibliography which simply cites EN 338 and EN 1994

Table 5.2. Contents of Part 1-2: *General – Structural fire design*

	Section	Description
1	General	Scope, references, definitions and symbols
2	Basis of design	Discusses requirements for load-bearing, separating and insulation functions, and gives methods for determining the design values of strength properties in an accidental fire situation
3	Material properties	Provides methods for determining the charring depth of timber members either exposed to or protected from fire, and gives general statements on the types of adhesive suitable for use in fire situations
4	Design procedures for mechanical resistance	Various design procedures including the stability of beams and columns, and formulae for determining the slip modulus of mechanically fastened joints in fire. The national annex states that of the two methods given for determining the strength of the residual section, the reduced cross-section method should be used
5	Design procedures for wall and floor assemblies	A couple of rules related to load-bearing and separating walls and floors respectively. (Rule 5.2(1) implies that external timber-frame walls should be designed to resist fire on both sides at the same time (which may lead to some debate!)
6	Connections	A very useful section which gives methods for calculating the period of fire resistance of various types of mechanical connection, both protected and unprotected from fire. It also covers connections made with internal and external steel plates, and axially loaded screws (e.g. supporting soffits)
7	Detailing	Requirements for minimum dimensions, the detailing of panel connections and the attachment of protective claddings
A	Parametric fire exposure	More information about charring depth and the fire resistance of members to edgewise bending
B	Advanced calculation methods	This annex concerns specialized fire design and is unlikely to be of general interest (e.g. ‘the effects of transient thermal creep should be taken into account … special attention should be given to transient states of moisture’)

C	Load-bearing floor joists and wall studs in assemblies whose cavities are completely filled with insulation	This useful annex includes methods for determining the residual strength and stiffness of wall studs and floor joists in assemblies exposed to fire on either one or both sides
D	Charring of members in wall and floor assemblies with void cavities	Additional information for calculating the time to start of charring and to failure of assemblies initially protected by gypsum plasterboard
E	Analysis of the separating function of wall and floor assemblies	This is a complicated design method for determining the time that separating walls and floors made with layers of various materials and thicknesses, including voids, will keep the temperature on the unexposed side to within specified limits. It could be useful for manufacturers who are developing new elements, but the performance of separating elements is normally determined by test
F	Guidance for users of this Eurocode Part	Two flowcharts to guide users through the fire design maze, one for load-bearing structural members and the other for the design of connections

Table 5.3. Contents of Part 2: *Bridges*

(Since this part of Eurocode 5 supplements Part 1-1 it is fairly short)

	Section	**Description**
1	General	Scope, references, definitions and symbols
2	Basis of design	A brief summary of the general Eurocode design basis, plus some additional load durations and partial factors for materials that are specific to bridge design. In particular Eurocode 5 Part 2 must be read in conjunction with the 26 pages of BS EN 1990 Annex 2: Application for bridges
3	Material properties	Requirements for pre-stressing steels
4	Durability	Various requirements and recommendations relevant to bridge design
5	Basis of structural analysis	Additional rules for the design of laminated deck plates (nail-, stress- and glued-laminated), composite members in general, and timber/composite members

Table 5.3. Contents of Part 2: *Bridges.* (*Contd*)

	Section	Description
6	Ultimate limit states	Formulae for calculating the strength of deck plates. Fatigue considerations
7	Serviceability limit states	Recommendations for limits on deflection and vibration in timber bridges
8	Connections with metal fasteners	Guidance for calculating the strength of connections between timber and concrete
9	Structural detailing and control	Two additional clauses relevant to bridge construction
A	Fatigue verification	Clause 6.2 of Eurocode 5 Part 2 requires a designer to verify that no failure or major damage due to fatigue will occur, but note 1 to the same clause states that a fatigue verification is not normally required for timber footbridges. Annex A provides a method for assessing whether a fatigue investigation is required and, if it is, a method for determining the design fatigue strength
B	Vibrations caused by pedestrians	Formulae for calculating the vertical and horizontal accelerations produced by one or more pedestrians crossing a bridge. The Annex does not provide any guidance for calculating the fundamental natural frequency of vibration in each direction, which must be calculated first

Key technical changes

The key technical changes from BS 5268 are:

a) the differentiation between ultimate, serviceability and accidental limit states;
b) the re-classification of load durations
c) the partial factor safety format, which requires safety factors to be applied manually to both loads and material properties, rather than having them all built into tabulated grade or basic values;
d) the use of load combination factors;
e) new symbols and material strength modification factors;
f) the use of Emean for all deflection calculations, choice of deflection limits, and the increased emphasis on calculating creep deflection;

g) the differentiation between principles and application rules (clauses which are designated by the letter 'P' are principles which must be complied with, whereas other clauses provide only recommended methods by which the principles may be satisfied);
h) BS EN 1995 is a theoretical design code rather than a code of best practice, so formulae replace tabulated values and most of the helpful advice given in BS 5268 has disappeared.

A few comments may be made on each of the above:

a) The differentiation between ultimate, serviceability and accidental limit states

Design work conforming to Eurocode 5 requires the calculation of at least two different design loads – one for strength and stability calculations (i.e. ULS or ultimate limit states), for which safety factors are applied to the loads, and the other for deflection calculations (i.e. SLS or serviceability limit states), for which safety factors are not used. As with BS 5268 design, if loads of more than one duration are applied to a member, a separate calculation of the ULS design load must be made for every load duration represented in order to determine the critical load combination.

Dealing with accidental limit states under Eurocode 5 is possibly more straightforward than with BS 5268 because it follows the same procedure as ultimate limit state design but with appropriately modified partial factors and load combination factors, and only one load duration normally has to be considered, that is instantaneous.

b) The reclassification of load durations

Whereas BS 5268 designated four different load durations (long-term, medium-term, short-term and very short-term), Eurocode 5 designates five, as shown in Table 5.4.

Thus the imposed floor load duration is reduced from long-term to medium-term, and the snow load duration is reduced from medium-term to short-term.

Table 5.4. Designations of load duration (UK national annex)

Designation	Definition	Examples of use
Permanent	More than 10 years	Self-weight
Long-term	6 months to 10 years	Storage loading (e.g. in lofts) and water tanks
Medium-term	1 week to 6 months	Imposed floor loading
Short-term	Less than 1 week	Snow, maintenance or man on roof, residual structure
Instantaneous	Instantaneous	Wind, impact, explosion

c) The partial factor safety format

Under BS 5268 design rules all the safety factors were incorporated in the tabulated material properties, which were applicable to BS 5268 long-term loading. For loadings of shorter duration the strength properties were increased by an appropriate load duration factor. In Eurocode design the tabulated material properties are 5% fractile characteristic test values which are applicable to test durations of only about 5 min. So the designer has to decrease them for loads of longer duration (short-term, medium-term, etc.), and then decrease them again by a material safety factor which depends on the type of timber material. For solid timber, a larger safety factor is required than for glulam or LVL because solid timber is more variable in its properties. In addition the characteristic loadings provided by BS EN 1991 must be increased by a safety factor which is larger for variable loads such as wind and snow than for permanent loads such as the weight of materials, since the former are known with less certainty.

d) The use of load combination factors

In common with the other material Eurocodes, load combination factors are applied when two or more variable loads act simultaneously. This can result in significantly more work for the designer, especially when different load durations are involved. For a roof truss with instantaneous wind, short-term snow, a medium-term point load on the ceiling tie, a long- term storage load and a permanent dead load, the resulting number of combinations for which a design load must be calculated is 11. And that is only for one possible position of a person!

e) New symbols and material strength modification factors

In most cases Eurocode 5 uses the symbol f for a strength property and σ for a stress. The symbol u is used for deformation in general (deflection or slip), w is used for deflection in particular, and W is used for the section modulus 1[1]. Most symbols have suffixes to identify the particular property to which they relate, for example $f_{c,0,k}$ is the characteristic compression strength parallel to the grain. The former x- and y-axes become y (major bending axis) and z (minor bending axis) respectively. The x-axis is now measured along the length of a beam or column. The BS 5268 factors K2 and K5, which modified strength for service class and duration of loading, have been confined into a single factor, k_{mod}, as illustrated in Table 5.5. A second factor, k_{ef}, is used to calculate creep deformation and joint slip, as shown in Table 5.6. The total number of material strength modification factors has been reduced from 59 in BS 5268 to 21 in Eurocode 5.

f) Deflections

In Eurocode 5 design, E_{mean} is used for all deflection calculations, whereas in BS 5268 design E_{min} (the 5% fractile value of the modulus of elasticity) was used for beams and laterally loaded columns unless they formed part of a load-sharing system.

Eurocode 0 (the popular name for BS EN 1990) states that ‘the serviceability criteria should be specified for each project and agreed with the client’. This gives a freedom of choice which can produce significant economies where relatively large deflections are tolerable, for example in the roof of a glulam sports hall. In BS 5268 design, a deflection limit of $0.003L$ (or the minimum of $0.003L$ and 14 mm for domestic floors) under full load was more or less normative for everything.

Creep deflection can be significant in timber materials. For example, Table 5.6 shows that in Service class 3 the total creep deflection during the 50 year design life of a solid timber beam is double its instantaneous elastic deflection under similar loading. Eurocode 5 suggests ranges of deflection limits for both instantaneous and final deflection. The Eurocodes do not require creep

---

[1] Since the symbol Z remains undefined in the Eurocode, British designers may choose to retain this for the section modulus.

Table 5.5. Values for k_{mod}

Material	**Service class**	**Load duration class**				
		Permanent	**Long**	**Medium**	**Short**	**Instantaneous**
Solid timber, glulam, LVL, plywood	1.2 3	0.6 0.5	0.7 0.55	0.8 0.65	0.9 0.7	1.1 0.9
OSB/3	1 2	0.4 0.3	0.5 0.4	0.7 0.55	0.9 0.7	1.1 0.9

Values are also given for other types of OSB and for various types of particleboard and fibreboard.

Table 5.6. Values for k_{def}

Material	Service class		
	1	2	3
Solid timber, glulam, LVL	0.6	0.8	2.00
Plywood, type EN 636-1 Plywood, type EN 636-2 Plywood, type EN 636-3	0.8 0.8 0.8	– 1.0 1.0	– – 2.5
OSB, type EN 300-3	1.5	2.25	–

Values are also given for other types of OSB and for various types of particleboard and fibreboard.

deflection to be calculated because the agreed deflection limits could be based on instantaneous deflection, which is the only kind of deflection for which an appropriate limit relating to plasterboard cracking has been investigated experimentally in the UK. However, because the UK national annex gives recommended deflection limits only for final deflection, most designers will feel obliged to include creep in their deflection calculations. Final deflection is obviously more relevant for appearance considerations if no brittle finishes are involved. The calculation of creep is important in the design of composite steel/timber components such as flitched beams, because as it progresses the timber becomes relatively less stiff, so the share of load taken by each material changes. Under BS 5268, the calculation of creep was optional, and indeed the elastic moduli tabulated for plywoods in BS 5258 already had creep taken into account.

g) The differentiation between principles and application rules

The Eurocodes permit designers to use alternative application rules to those provided, provided that these conform to the relevant principles and produce equivalent levels of safety, serviceability and durability as those produced by the recommended application rules (BS EN 1990 Clause 1.4.5.). However, regulatory bodies may require proof of such conformance, in which case it may be simpler to use the application rules provided by the Eurocodes.

h) BS EN 1995 is a theoretical design code

The theoretical nature of Eurocode 5 provides one of the major challenges to timber designers who are more familiar with BS 5268. Since it is almost entirely formula-based there are no tables of pre-calculated fastener loads or permissible trussed rafter spans, no bracing solutions for standard trussed rafter roofs and no masonry wind-shielding factors for timber-frame buildings. The formulae for connections are very complicated, making almost essential the use of dedicated spreadsheets or computer programs. Eurocode 5 provides no tables of properties for particular species of timber or types of plywood: solid timber must be assigned to a strength class for which values are provided in a separate standard, BS EN 338, and for plywood characteristic test values must be obtained from the manufacturer. No guidance is provided for the design of glued joints, for which values have to be obtained from tests.

Principal benefits

The new code does, however, offer some significant benefits:

- more consistent levels of reliability;
- multinational companies will benefit by being able to use the same timber design code in many different countries both within and outside Europe, see [1];
- using a similar design format to that used for other structural materials should help to make timber design more accessible, and make it easier for timber to be incorporated into structural analysis programs for steel and concrete;
- the separation of ultimate and serviceability design states permits the use of more rational design limits
- the separation of principles and application rules allows the engineer more freedom but requires more understanding on his or her part;
- the direct use of characteristic test values simplifies the adoption of new timber materials and components;
- the connection design formulae can cater for LVL, OSB and particleboard as well as for solid timber materials;
- the dedicated timber bridge design code should encourage the use of timber in lightweight bridges;
- the formulaic approach facilitates the development of spreadsheets and software.

Effects on the timber industry

Major changes in timber usage and specification are unlikely. However:

- characteristic strength properties for panel products and components such as timber I-joists and metal hardware must now be obtained in accordance with CEN testing standards;
- floors may have to be a little stiffer (i.e. more timber);
- large roof structures without brittle finishes may not require so much timber;
- there will have to be yet more reliance on software for the design of trussed rafters, connections and timber frame walls.

General guidance and publications

Various manuals and guidance documents have been published in the UK, and many of these are listed on the Eurocodes Expert website [2] under 'Timber/ Publications'. Additionally BSI intends to preserve the guidance in BS 5268 which would otherwise be lost by producing a new publication, PD 6693, *Complementary information for use with Eurocode 5*. A design example is given in Annex A. The Institution of Structural Engineers have published a manual for the design of timber building structures to Eurocode 5 [3].

Summary

With supporting information, Eurocode 5 is a workable design code that is particularly useful for multinational companies and the designers of larger engineering structures and bridges.

References

[1] The Structural Engineer, 18 September 2007. The Civil Engineer Center for Integrating Information (www.thestructuralengineer.info)

[2] http://www.eurocodes.co.uk

[3] *Manual for the design of timber building structures to Eurocode 5*, The Institution of Structural Engineers/TRADA. December 2007

Eurocode 6: Design of masonry structures

Professor John Roberts, Principal, Technical Innovation Consultancy

Eurocode 6 (BS EN 1996) follows the general presentation of the material Eurocodes in that Part 1-1 covers the design of plain and reinforced masonry whilst Part 1-2 deals with structural fire design. There are two further parts, Part 2 which deals primarily with the selection of materials and execution of masonry and Part 3 which covers simplified calculation methods for unreinforced masonry structures.

Eurocode 6 has been developed to enable the designer to use the following types of masonry unit: clay, calcium silicate, aggregate concrete, autoclaved aerated concrete (Aircrete), manufactured stone and natural stone. European Standards for these materials have now been published by BSI and form part of an array of standards relating to masonry produced under the auspices of CEN TC 125.

Eurocode 6 therefore comprises the following parts:

- EN 1996-1-1, *Common rules for reinforced and unreinforced masonry structures*;
- EN 1996-1-2, *Structural fire design*;
- EN 1996-2, *Design, selection of materials and execution of masonry*;
- EN 1996-3, *Simplified calculation methods*;

Each Part has a corresponding national annex.

The standards supporting Eurocode 6 were developed within a common framework but it has not proved possible to standardize all the test methods used by the different materials. Words like 'brick' and 'block' have disappeared from the European vocabulary and they are all referred to as masonry units. Well-established products such as Engineering bricks now need to be specified by their performance requirements.

The standards supporting the use of masonry in Eurocode 6 were introduced by BSI and required, as an interim measure, the updating of the three Parts of BS 5628 to accommodate the revised material standards and test methods. These new Parts of BS 5628 were published at the end of 2005, two of the key factors that changed being:

- the six new masonry unit standards each introduced new methods for determining the compressive strength of masonry units;
- the method of determining the characteristic compressive strength of masonry changed from testing storey height panels to much smaller masonry wallette specimens.

Ancillary components are now dealt with in a more coherent way and suitable values of partial safety factors have been introduced. The partial safety factors for use with masonry are given in National Annex Table NA.1 and shown here in Table 6.1. Two levels of attestation of conformity are recognized, category I and category II, and this will be declared by the manufacturer of the masonry units. Two classes of execution control are recognized, 1 and 2.

BS 5628 was the first limit state design code for masonry in the world and UK designers are very familiar with the principles which have now been encapsulated in Eurocode 6. There are, however, a few major changes which UK designers will need to become familiar with.

During the drafting of Eurocode 6, a way had to be found to deal with the wide range of masonry units used across Europe. This range not only includes different material such as clay, concrete and stone, but also a variety of configurations based upon the proportion and direction of any holes or perforations, web thickness etc. This has resulted in four grouping of masonry units according to the percentage size and orientation of holes in the units when laid. Historically the UK only has experience of Group 1 and Group 2 masonry units and therefore has no national database on which to base structural performance. In the UK national annex, therefore, information is only provided for Group 1 and Group 2 units and properties for other Groups would normally need to be established by testing.

Two levels of quality assurance for the manufacture of masonry units have been retained in the UK. These are now described as follows.

- Category I masonry units which have a declared compressive strength with a probability of failure to reach it not exceeding 5%.
- Category II masonry units which are not intended to comply with the level of confidence of Category I units.

Table 6.1. Value of partial safety factors for materials for ultimate limit states

Class of execution control:	γ_M 1[A)]	2[A)]
Material		
Masonry		
When in a state of direct or flexural compression		
Unreinforced masonry made with:		
units of category I	2.3[B)]	2.7[B)]
units of category II	2.6[B)]	3.0[B)]
Reinforced masonry made with:		
units of category I	2.0[B)]	[C)]
units of category II	2.3[B)]	[C)]
When in a state of flexural tension		
units of category I and II	2.3[B)]	2.7[B)]
When in a state of shear		
Unreinforced masonry made with:		
units of category I and II	2.5[B)]	2.5[B)]
Reinforced masonry made with:		
units of category I and II	2.0[B)]	[C)]
Steel and other components		
Anchorage of reinforcing steel	1.5[D)]	[C)]
Reinforcing steel and prestressing steel	1.15[D)]	[C)]
Ancillary components – wall ties	3.5[B)]	3.5[B)]
Ancillary components – straps	1.5[E)]	1.5[E)]
Lintels in accordance with EN 845-2	See national annex to BS EN 845-2	See national annex to BS EN 845-2

Refer to the UK National Annex Table NA.1 for details of the footnotes.

The characteristic compressive strength of masonry is no longer presented in the form of tables but as an equation. This equation includes the normalized strength of the masonry and the strength of the mortar. The normalized strength is new to the UK and relates the compressive strength of the unit determined by test to a standardized shape and moisture content. The normalized compressive strength is the compressive strength of the units converted to the air dried compressive strength of an equivalent 100 mm wide by 100 mm high masonry unit. The detail is contained in the test methods for masonry units in accordance with EN 772-1. The advantage to the designer is that the normalized strength is independent of the size of the units used in the final construction thereby obviating the need for recalculation.

The characteristic compressive strength of masonry (other than shell bedded masonry) is determined from the results of tests in accordance with EN 1052-1. The tests are carried out on small wallette specimens rather than the storey height panels used in BS 5628. The designer has the option of either having the units intended to be used in a project tested or to use the values determined from the UK national database. The latter values are provided in the UK national annex in the form of the constants to be used in the following equation:

$$f_k = K\, f_b^{\alpha} f_m^{\beta} \qquad \text{[Equation (3.1) of BS EN 1996]}$$

where

f_k is the characteristic compressive strength of the masonry, in N/mm^2
K is a constant
α, β are constants
f_b is the normalized mean compressive strength of the units, in the direction of the applied action effect, in N/mm^2
f_m is the compressive strength of the mortar, in N/mm^2

Values of K to be used with equation 3.1 are provided in the UK National Annex to BS EN 1996-1-1 (Table NA.4) and are shown in Table 6.2.

The value of K is reduced when a mortar joint runs continuously or intermittently through the masonry at right angles to the cross joints. Note that for blocks laid flat the table contains a specific value for K to be used in equation 3.1.

Values of α, β for use with equation 3.1 are shown in Table 6.3.

There are a number of limitations placed on equation 3.1 which are detailed in the UK national annex.

The designation of mortars has also changed with the need for a declaration based on strength rather than mix proportions. Thus an M12 mortar may be expected to have a strength of 12 N/mm^2. Equivalent mixes are shown in National Annex Table NA.2 and are shown in Table 6.4.

Table 6.2. Values of *K* to be used with equation 3.1

Masonry unit		General purpose mortar	Thin layer mortar (bed joint ≥ 0.5 mm and ≤ 3 mm)	Lightweight mortar of density	
				$600 \leq \rho_d \leq 800$ kg/m^3	$800 < \rho_d \leq 1300$ kg/m^3
Clay	Group 1	0.50	0.75	0.30	0.40
	Group 2	0.40	0.70	0.25	0.30
	Group 3	[A]	[A]	[A]	[A]
	Group 4	[A]	[A]	[A]	[A]
Calcium silicate	Group 1	0.50	0.80	[B]	[B]
	Group 2	0.40	0.70	[B]	[B]
Aggregate concrete	Group 1	0.55	0.80	0.45	0.45
	Group 1[C] (units laid flat)	0.50	0.70	0.40	0.40
	Group 2	0.52	0.76	0.45	0.45
	Group 3	[A]	[A]	[A]	[A]
	Group 4	[A]	[A]	[A]	[A]
Autoclaved aerated concrete	Group 1	0.55	0.80	0.45	0.45
Manufactured stone	Group 1	0.45	0.75	[B]	[B]
Dimensioned natural stone	Group 1	0.45	[B]	[B]	[B]

[A] Group 3 and 4 units have not traditionally been used in the UK, so no values are available.
[B] These masonry unit and mortar combinations have not traditionally been used in the UK, so no values are available.
[C] If Group 1 aggregate concrete units contain formed vertical voids, multiply K by $(100 - n)/100$, where n is the percentage of voids, maximum 25%.

Table 6.3. Values to be used in equation 3.1

Type of mortar	Values to be used
General purpose mortar	$\alpha = 0.7$ and $\beta = 0.3$
Lightweight mortar	$\alpha = 0.7$ and $\beta = 0.3$
Thin layer mortar in bed joints of thickness 0.5 mm to 3 mm. Using clay units of Group 1, calcium silicate units, aggregate concrete units and autoclaved aerated concrete units	$\alpha = 0.85$ and $\beta = 0$
Thin layer mortar in bed joints of thickness 0.5 mm to 3 mm. Using clay units of Group 2	$\alpha = 0.7$ and $\beta = 0$

Table 6.4. Acceptable assumed equivalent mixes for prescribed masonry mortars

Compressive strength class[A)]	Prescribed mortars (proportion of materials by volume) (see Note)				Mortar designation
	Cement[B)]: lime: sand with or without air entrainment	Cement[B)]: sand with or without air entrainment	Masonry cement[C)]: sand	Masonry cement[D)]: sand	
M12	1:0 to ¼:3	1:3	Not suitable	Not suitable	(i)
M6	1:½:4 to 4½	1:3 to 4	1:2½ to 3½	1:3	(ii)
M4	1:1:5 to 6	1:5 to 6	1:4 to 5	1:3½ to 4	(iii)
M2	1:2:8 to 9	1:7 to 8	1:5½ to 6½	1:4½	(iv)

A) The number following ‘M’ is the compressive strength at 28 days in N/mm^2.
B) Cement or combinations of cement in accordance with **NA.2.3.2**,Except masonry cements.
C) Masonry cement in accordance with **NA.2.3.2**, (inorganic filler other than lime).
D) Masonry cement in accordance with **NA.2.3.2** (lime).
NOTE When the sand portion is given as, for example, 5 to 6, the lower figure should be used with sands containing a higher proportion of fines whilst the higher figure should be used with sands containing a lower proportion of fines.

A further area of change for vertical load relates to the treatment of eccentricity where a frame analysis approach is implied rather than the BS 5628 approach of assuming that any eccentricity at the top of the wall reduces to zero at the bottom of the wall. The use of a frame analysis will not usually be justified given typical UK construction practice and it is still acceptable to use the BS 5628 approach. The concept of an initial eccentricity to allow for any inaccuracies in the construction of the masonry is also introduced.

The effective height of a masonry wall is obtained by applying a factor to the clear height of the wall such that:

$$h_{ef} = \rho_n h$$

where

- h_{ef} is the effective height of the wall
- h is the clear storey height of the wall
- ρ_n is a reduction factor, where n = 2, 3 or 4, depending upon the edge restraint or stiffening of the wall. The reduction factor to be applied depends upon the restraint offered by adjoining elements but UK designers will find that the reductions obtained are similar to BS 5628.

The value for the minimum thickness, t_{min}, of a load-bearing wall should be taken as 90 mm for a single leaf wall and 75 mm for the leaves of a cavity wall. For a single leaf-wall, a double-leaf wall, a faced wall, a shell-bedded wall and a grouted cavity wall the effective thickness (t_{ef}) is taken as the actual thickness of the wall (t).

When a wall is stiffened by piers, the effective thickness is enhanced using similar factors to those contained in BS 5628.

For a cavity wall the effective thickness in the UK is determined using the following equation:

$$t_{ef} = \sqrt[3]{t_1^3 + t_2^3}$$

where

- t_1 is the effective thickness of the of the outer or unloaded leaf
- t_2 is the effective thickness of the of the inner or loaded leaf

Note that the effective thickness of the unloaded leaf should not be taken to be greater than the thickness of the loaded leaf and that ties should be provided at 2.5 per m^2.

The slenderness ratio of the wall is obtained by dividing the effective height by the effective thickness and should not be greater than 27 for walls subjected to mainly vertical loading.

When a wall is subjected to vertical loads which result in an eccentricity at right angles to the wall, Eurocode 6 requires the resistance of the wall to be checked at the top, mid-height and bottom. The eccentricity of the wall at the top and bottom includes an initial eccentricity, e_{init}, for construction imperfections as well as e_{he}, the eccentricity at the top or bottom of the wall, resulting from the applied loads.

Hence:

$$e_i = \frac{M_{id}}{N_{id}} + e_{he} + e_{init} \geq 0.05t$$

where:

M_{id} is the design value of the bending moment at the top or the bottom of the wall resulting from eccentricity of the floor load at the support

N_{id} is the design value of the vertical load at the top or the bottom of the wall.

The eccentricity at the mid-height of the wall, e_{mk}, includes the initial eccentricity, e_{init}, the lateral load eccentricity, e_{hm}, and the load eccentricity, e_m. In the UK, creep eccentricity, e_k, is ignored if the slenderness ratio is not greater than 27. e_{init} may be taken as $h_{ef}/450$.

The mid-height eccentricity, e_{mk}, is:

$$e_{mk} = e_m + e_k$$

which must be greater or equal to $0.05t$

$$e_m = \frac{M_{md}}{N_{md}} + e_{hm} + e_i$$

where:

M_{md} is the design value of the greatest moment at the middle height of the wall resulting from the moments at the top and bottom of the wall, including any load applied eccentrically to the face of the wall.

N_{md} is the design value of the vertical load at the middle height of the wall, including any load applied eccentrically to the face of the wall.

At the top or bottom of the wall the reduction factor for slenderness and eccentricity is given by:

$$\Phi_i = 1 - 2\frac{e_i}{t}$$

where:

Φ_i is the reduction factor at the top or bottom of the wall
e_i is the eccentricity at the top or bottom of the wall
t is the thickness of the wall

In the middle of the wall the capacity reduction factor may be determined by using either the equation or the graph given in Annex G of BS EN 1996-1-1. The value of the modulus of elasticity to be taken in the UK is $1000f_k$.

The design resistance of a single leaf wall per unit length, N_{Rd}, is given by the following:

$$N_{Rd} = \Phi t f_d$$

where:

Φ is a capacity reduction factor allowing for the effects of slenderness and eccentricity of loading (the least favourable value obtained for the top and bottom and mid-height).
t is the thickness of the wall
f_d is the design compressive strength of the masonry

For sections of small plan area, less than $0.1\,m^2$, f_d should be multiplied by

$$0.7 + 3A$$

where

A is the load-bearing horizontal cross-sectional area of the wall in m^2.

Concentrated loads are dealt with by a calculation approach rather than the use of the bearing types shown in BS 5628. The Eurocode uses a more conservative dispersion angle of 60°.

For a Group 1 unit (not shell bedded) the vertical load resistance is given by:

$$N_{Rdc} = \beta A_b f_d$$

where:

$$\beta = \left(1 + 0.3\frac{a_1}{h_c}\right)\left(1.5 - 1.1\frac{A_b}{A_{ef}}\right)$$ (where A_b/A_{ef} is not to be taken as greater than 0.45)

which should not be less than 1.0 nor taken to be greater than:

$$1.25 + \frac{a_1}{2h_c} \text{ or } 1.5$$ (whichever is the lesser)

where:

β is an enhancement factor for load
a_1 is the distance from the end of the wall to the nearer edge of the loaded area
h_c is the height of the wall to the level of the load
A_b is the loaded area
A_{ef} is the effective area of the bearing $l_{efm}t$
l_{efm} is the effective length of the bearing as determined at the mid-height of the wall or pier
t is the thickness of the wall, taking into account the depth of recesses in joints greater than 5 mm.

For walls built with Groups 2, 3 and 4 masonry units and when shell bedding is used it is necessary to check that, locally under the bearing of a concentrated load, the design compressive stress does not exceed the design compressive strength of the masonry, f_d (i.e. β is taken to be 1.0).

In any case the eccentricity of the load from the centre line of the wall should not be greater than $t/4$.

Eurocode 6 offers two approaches to the design of laterally loaded panels. The first method relies on the flexural strength of the masonry and makes use of yield line analysis. The second method is an approach based on arching and the assumption of a three-pinned arch being formed within the wall.

The flexural strength approach is the most widely used and does not depend upon rigid supports to resist arch thrust. In the UK the reliance on the development of tensile strength in the masonry has meant that this design approach has usually been limited to transitory loads only. Eurocode 6 indicates that the flexural strength of masonry should not be used in the design of walls subjected to lateral earth pressure.

The characteristic shear strength of masonry is a function of the characteristic initial shear strength of the masonry and the design compressive stress orthogonal to the shear plane being considered. The values of the initial shear strength of masonry are given in Table NA.5 and shown in Table 6.5.

The characteristic shear strength is given by the following relationships:

$f_{vk} = f_{vko} + 0.4\sigma_d$ but not greater than $0.065f_b$ for fully-filled mortar joints

$f_{vk} = 0.5f_{vko} + 0.4\sigma_d$ but not greater than $0.045f_b$ for unfilled perpend joints

where:

f_{vk} is the characteristic shear strength of masonry

f_{vko} is the characteristic initial shear strength of masonry, under zero compressive stress

σ_d is the design compressive stress perpendicular to the shear in the member at the level under consideration, using the appropriate load combination based on the average vertical stress over the compressed part of the wall that is providing shear resistance

f_b is the normalized compressive strength of the masonry units (as described in clause 3.1.2.1 of BS EN 1996-1-1) for the direction of application of the load on the test specimens being perpendicular to the bed face.

BS EN 1996-1-1 contains a limited amount of information relating to the design of reinforced masonry and no detailed information on the design of prestressed masonry. The clauses relating to reinforced masonry contain information similar to that provided in BS 5628-2 and Annex J permits enhancement to shear based on BS 5628-2. However, it does not provide the coherent design thread provided by BS 5628-2.

BS EN 1996-1-2 provides information on the passive fire resistance of masonry walls so that the designer can ensure that the load-bearing performance is maintained for the necessary period of time and that the fire is appropriately contained.

Fire design largely remains in the form of tables similar to those contained in BS 5628-3. The fire resistance of a load-bearing wall now comprises two values depending upon how highly loaded the wall is and is further enhanced if the wall is plastered. Most designers will find that the tabulated data covers most situations but there is also the provision for testing. The tables cover load-bearing and non-load-bearing walls, single leaf, cavity and separating walls.

Table 6.5. Values of the initial shear strength of masonry, f_{vko}

Masonry units	Strength class of general purpose mortar	f_{vko} (N/mm^2)		
		General purpose mortar	Thin layer mortar (bed joint ≤ 0.5 mm and ≥ 3 mm)	Lightweight mortar
Clay	M12	0.30	0.30	0.15
	M4 and M6	0.20		
	M2	0.10		
Calcium silicate	M12	0.20	0.40	0.15
	M4 and M6	0.15		
	M2	0.10		
Aggregate concrete, autoclaved aerated concrete, manufactured stone and dimensioned natural stone	M12	0.20	0.30	0.15
	M4 and M6	0.15		
	M2	0.10		

BS EN 1996-2 gives basic rules for the selection of materials and execution to enable it to comply with the other parts of BS EN 1996. It deals with the following aspects:

- the selection of masonry materials;
- factors affecting the performance and durability of masonry;
- resistance of buildings to moisture penetration;
- storage, preparation and use of materials on site;
- the execution of masonry;
- masonry protection during execution.

The micro conditions of exposure to which completed masonry is subjected are divided into five main classes, namely:

- MX1 – in a dry environment;
- MX2 – exposed to moisture or wetting;
- MX3 – exposed to moisture or wetting plus freeze/thaw cycling;
- MX4 – exposed to saturated salt air or seawater;
- MX5 – in an aggressive chemical environment.

BS EN 998-2 defines the durability classes for mortar. These may be expressed as follows:

- P – mortar for use in masonry subjected to passive exposure;
- M – mortar for use in masonry subjected to moderate exposure;
- S – mortar for use in masonry subjected to severe exposure.

These designations may also be used for site-made mortar but they are not usable until the definitions are clarified in PD 6697.

The information contained in BS EN 1996-2 is not nearly as extensive as that contained in BS 5628-3. The key parts of BS 5628-3 not covered by the Eurocode will be published by BSI in PD 6697.

BS EN 1996-3 deals with simplified calculation methods for unreinforced masonry but it is not anticipated that this will be widely used in the UK where other guidance, for example Approved Document A [1] of the Building Regulations for England and Wales, is likely to produce more cost effective outcomes.

Reference

[1] Approved Document A, The Building Regulations England and Wales.

Eurocode 7: Geotechnical design

Andrew Harris, Director, and Dr Andrew Bond, Director, Geomantix Ltd.

Overview

Eurocode 7 covers geotechnical design and is provided in two parts: Part 1 – *General rules* (BS EN 1997-1) and Part 2 – *Ground investigation and testing* (BS EN 1997-2).

Part 1 is divided into twelve clauses and nine annexes, as shown in Figure 7.1. In this diagram, the size of each segment of the pie is proportional to the number of paragraphs in the relevant section.

Part 1 provides a general framework for geotechnical design, definition of ground parameters, characteristic and design values, general rules for site

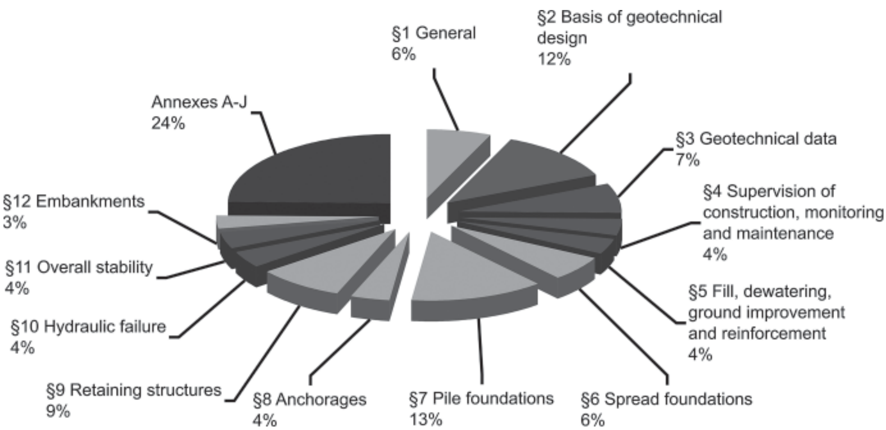


Figure 7.1. Contents of Eurocode 7 Part 1 (Bond and Harris 2008)

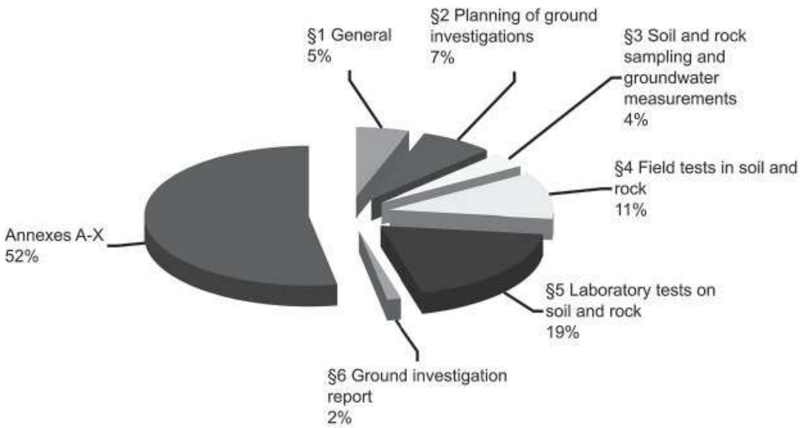


Figure 7.2. Contents of Eurocode 7 Part 2 (Bond and Harris 2008)

investigation, rules for the design of the main types of geotechnical structures, and some assumptions on execution procedures.

Part 2 is divided into six sections and twenty-four annexes, as illustrated in Figure 7.2.

Part 2 provides detailed rules for site investigations, general test specifications, derivations of ground properties and the geotechnical model of the site, and examples of calculation methods based on field and laboratory testing.

Eurocode 7 Part 1 *General rules*

Table 7.1 shows both the common (white background) and different (grey) sub-section headings in clauses 6–9 and 11–12 of EN 1997-1, dealing with the rules for geotechnical structures – each present the information in a different manner reflecting the authorship of these sections. This difference in authorship has led to inconsistencies in the sub-section headings for each geotechnical structure and to the associated level of detail. For example, 6.5 for spread foundations provides a relatively short section on ultimate limit state design with no illustrations, whereas this topic is covered in detail in 9.7 for retaining structures with a large number of diagrams.

Table 7.1. Common and different sub-section headings in EN 1997-1

Sub-section[1)]	Common to clauses 6–9, 11 and 12	Clause 6 Spread foundations	Clause 8 Anchorages	Clause 7 Pile foundations	Clause 9 Retaining structures
§x.1	General				
§x.2	Limit states				
§x.3	Actions, geometrical data, and design situations (or variations on this title)				
§x.4	Design methods and construction considerations (or variations on this title)				
§x.5	Ultimate limit state design			Pile load tests	
§x.6	Serviceability limit state design			Axially loaded piles	Water pressures
§x.7	Supervision and monitoring	Foundations on rock; additional design considerations	Suitability tests	Transversely loaded piles	Ultimate limit state design
§x.8	–	Structural design of spread foundations	Acceptance tests	Structural design of piles	Serviceability limit state design
§x.9	–	Supervision and monitoring (or variations on this title)			(missing)

1) The symbol '§x' is meant to be used interchangeably to represent clause numbers 6 to 9, 11 and 12.

Design requirements

Perhaps the most significant requirement of Eurocode 7 is the following commitment to limit state design: 'For each geotechnical design situation it shall be verified that no relevant limit state … is exceeded.' [EN 1997-1 2.1(1)P]

For many geotechnical engineers across Europe, this represents a major change in design philosophy. Traditional geotechnical design, using lumped factors of safety, has proved satisfactory over many decades and much experience has been built on such methods. However, the use of a single factor to account for all uncertainties in the analysis – although convenient – does not provide a proper control of different levels of uncertainty in various parts of the calculation. A limit state approach forces designers to think more rigorously about possible modes of failure and those parts of the calculation process where there is most uncertainty. This should lead to more rational levels of reliability for the whole structure. The partial factors in Eurocode 7 have been chosen to give similar designs to those obtained using lumped factors – thereby ensuring that the wealth of previous experience is not lost by the introduction of a radically different design methodology.

Limit state philosophy has been used for many years in the design of structures made of steel, concrete and timber. Where these structures met the ground was, in the past, a source of analytical difficulties. The Eurocodes present a unified approach to all structural materials and should lead to less confusion and fewer errors when considering soil–structure interaction.

Complexity of design

A welcome requirement of Eurocode 7 is the mandatory assessment of risk for all design situations: 'the complexity of each geotechnical design shall be identified together with the associated risks … a distinction shall be made between light and simple structures and small earthworks … with negligible risk [and] other geotechnical structures.' [EN 1997-1 2.1(8)P]

The idea here is that when negligible risk is involved, the design may be based on past experience and *qualitative* geotechnical investigations. In all other cases, *quantitative* investigations are required. There are many schemes for assessing risk that may be used in conjunction with a Eurocode 7 design. For example, the approach outlined in the UK Highways Agency's document HD22/023 requires designers to identify possible hazards for a project or operation within that project. Such risk assessments clearly satisfy the requirements of Eurocode 7 to identify the complexity and risks of geotechnical design.

Geotechnical categories

To assist geotechnical engineers in classifying risk, Eurocode 7 introduces three geotechnical categories, their design requirements, and the design procedure they imply – as summarized in Table 7.2. The geotechnical categories are defined in a series of application rules, not principles, and hence alternative methods of assessing geotechnical risk could be used.

The design requirements and procedure for GC3 warrant comment, since it is not immediately clear what 'include alternative provisions and rules' means. Eurocode 7 does not say 'use alternative principles and rules' (even though the word 'provisions' could be taken to include the principles). It is considered that designs for GC3 structures should follow the principles of Eurocode 7, but the application rules provided in the standard may not be sufficient on their own to satisfy those principles. Hence alternative (and/or additional) rules may be required.

It is not necessary to classify all parts of a project in one geotechnical category. Indeed, many projects will comprise a mixture of GC1 and GC2 (and in some cases GC3) elements.

Eurocode 7 gives examples of structures in the three geotechnical categories, as shown in Table 7.2. A structure's geotechnical category influences the level of supervision and monitoring called for in Section 4 of EN 1997-1.

Structures in GC1 can safely be designed and built based on past experience, because they involve no unusual features or circumstances. The examples in GC2 form the bread and butter work of many geotechnical design offices, requiring thoughtful but not unusual design and construction. However, the design of structures in GC3 requires careful thought because of their exceptional nature; the keywords are 'large or unusual', 'abnormal or exceptionally difficult', 'highly seismic', 'probable instability', etc.

Table 7.2. Geotechnical categories and their requirements

<table>
<tr><th>GC</th><th>Includes …</th><th>Design requirements</th><th>Design procedure</th></tr>
<tr><td>1</td><td>Small and relatively simple structures … with negligible risk</td><td>Negligible risk of instability or ground movements; ground conditions are ‘straightforward’; no excavation below water table (or such excavation is ‘straightforward’)</td><td>Routine design and construction (i.e. execution) methods</td></tr>
<tr><td colspan="4">No examples given in EN 1997-1.</td></tr>
<tr><td colspan="4">Investigation requirements: normally limited (local knowledge and experience often sufficient)</td></tr>
<tr><td>2</td><td>Conventional types of structure and foundation with no exceptional risk or difficult soil or loading conditions</td><td>Quantitative geotechnical data and analysis to ensure fundamental requirements are satisfied</td><td>Routine field and lab testing
Routine design and execution</td></tr>
<tr><td colspan="4">Examples: spread, raft and pile foundations; walls and other structures retaining or supporting soil or water; excavations; bridge piers and abutments; embankments and earthworks; ground anchors and other tie-back systems; tunnels in hard, non-fractured rock, not subject to special water-tightness or other requirements</td></tr>
<tr><td colspan="4">Investigation requirements: provisions of EN 1997-2 apply</td></tr>
<tr><td>3</td><td>Structures or parts of structures not covered above</td><td colspan="2">Include alternative provisions and rules to those in Eurocode 7</td></tr>
<tr><td colspan="4">Examples: very large or unusual structures; structures involving abnormal risks or unusual or exceptionally difficult ground or loading conditions; structures in highly seismic areas; structures in areas of probable site instability or persistent ground movements that require separate investigation or special measures</td></tr>
<tr><td colspan="4">Investigation requirements: number of investigations at least the same as for Category 2 projects. The circumstances that place a project in Category 3 may necessitate additional investigations and more advanced tests</td></tr>
</table>

Limit states

When designing a geotechnical structure, the engineer needs to identify the possible ultimate and serviceability limit states that are likely to affect the structure. Ultimate limit states are those that will lead to failure of the ground or the structure; serviceability limit states are those that result in unacceptable levels of deformation, vibration, noise, or flow of water or contaminants.

Eurocode 7 identifies five categories of ultimate limit states for which different sets of partial factors are provided: failure or excessive deformation in the ground (GEO); internal failure or excessive deformation of the structure (STR); loss of static equilibrium (EQU); loss of equilibrium or excessive deformation due to uplift (UPL); hydraulic heave, piping and erosion (HYD).

The following ultimate limit states should be checked for all geotechnical structures: loss of overall stability (of the ground and/or associated structures); combined failure in the ground and structure; and structural failure due to excessive ground movement. For serviceability limit states the following should be checked: excessive settlement; excessive heave; and unacceptable vibrations.

Verification of the serviceability limit state is a key requirement of a Eurocode 7 design, which does not explicitly feature in more traditional approaches. In traditional design it is assumed in many calculation procedures that the lumped factor of safety provides not only safety against failure but also will limit deformations to tolerable levels. For example, it is common to adopt factors of safety against bearing failure of shallow foundations in excess of 3.0. This is far greater than needed to ensure sufficient reserve against failure. Eurocode 7 generally requires a specific check that serviceability limits are met and thus serviceability requirements are more likely to be the governing factor for many settlement sensitive projects.

Design situations

Design situations have a key role to play in the selection of actions to include in design calculations and in the choice of partial factors to apply to both actions and material properties. Design situations are: ‘Sets of physical conditions representing the real conditions occurring during a certain time interval for which the design will demonstrate that relevant limit states are not exceeded.’ [EN 1990 1.5.2.2]

Table 7.3. Common geotechnical problems

Design situation	Real conditions	Term	Example
Persistent	Normal	Long	Buildings and bridges founded on coarse soils and fully-drained fine soils
		Short	Partially-drained slope in fine soils (with DWL less than 25 years)
Transient	Temporary	Long	Temporary works in coarse soils
		Short	Temporary works in fine soils
Accidental	Exceptional	Long	Buildings and bridges founded on coarse soils and quick-draining fine soils
Seismic		Short	Buildings and bridges founded on slow-draining fine soils

Eurocode 7 requires consideration of both short- and long-term design situations, to reflect the sometimes vastly different resistances obtained from drained and undrained soils. At first sight, the requirements of EN 1997-1 appear to cut across those of EN 1990. However, it is not difficult to combine these ideas to cater for common geotechnical problems (see Table 7.3).

Geotechnical actions

EN 1997-1 lists twenty different types of action that should be included in geotechnical design. These include obvious things, such as the weight of soil, rock and water; earth pressures and ground-water pressures; and removal of load or excavation of ground – and less obvious things, such as movements caused by caving; swelling and shrinkage caused by climate change; and temperature effects, including frost action.

Distinction between favourable and unfavourable actions

The Eurocodes make an important distinction between favourable (or stabilizing) and unfavourable (destabilizing) actions, which is reflected in the

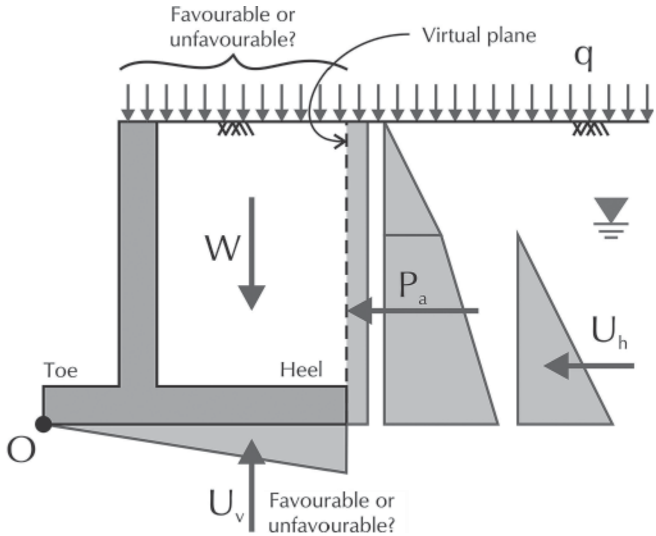


Figure 7.3. Examples of favourable and unfavourable actions (Bond and Harris 2008)

values of the partial factors γ_F applied to each type of action. Unfavourable/destabilizing actions are typically increased by the partial factor (i.e. $\gamma_F > 1$) and stabilizing actions are decreased or left unchanged (i.e. $\gamma_F \leq 1$).

Consider the design of the T-shaped gravity retaining wall shown in Figure 7.3. To provide sufficient reliability against bearing capacity failure, the self-weight of the wall and the soil on top of its heel (W) should be treated as unfavourable (since they increase the effective stress beneath the wall base) – but as favourable for sliding and overturning. The imposed surcharge q is unfavourable for bearing, sliding and overturning where it acts to the right of the virtual plane shown in Figure 7.3. But where it extends to the left of the virtual plane, it has the same effect as the wall's self-weight W.

Consider next the vertical and horizontal thrusts U_v and U_h owing to ground water pressure acting on the wall's boundaries. The horizontal thrust U_h is unfavourable for bearing, sliding and overturning, whereas the vertical thrust U_v is favourable for bearing (since it helps to counteract W), but unfavourable for sliding (reducing the effective stress beneath the wall base) and overturning (since it increases the anticlockwise overturning moment about 'O').

But it is illogical to treat an action as both favourable and unfavourable in the same calculation. Eurocode 7 deals with this issue in what has become known as the 'Single-Source Principle': 'Unfavourable (or destabilizing) and favourable (or stabilizing) permanent actions may in some situations be considered as coming from a single source. If … so, a single partial factor may be applied to the sum of these actions or to the sum of their effects.' [EN 1997-1 2.4.2(9)P NOTE]

This note allows the thrusts U_h and U_v to be treated in the same way – either both unfavourable or both favourable, whichever gives the more onerous design condition.

The Single-Source Principle has a profound effect on the outcome of some very common design situations. It also precludes the use of submerged weights in design calculations since, by replacing the gross weight W and water thrust U_v in Figure 7.3 by the submerged weight $W' = W - U_v$, the choice is implicitly made to treat both the self-weight and the water thrust in the same way.

Geotechnical design

Driscoll et al (2007) [1] state that:

> *The Eurocodes adopt, for all civil and building engineering materials and structures, a common design philosophy based on the use of separate limit states and partial factors, rather than 'global' factors (of safety); this is a substantial departure from much traditional geotechnical design practice … an advantage of BS EN 1997-1 is that its design methodology is largely identical with that for all of the structural Eurocodes, making the integration of geotechnical design with structural design more rational.*

The basic requirements of design are that for ultimate limit states the design effect of the actions, E_d, must less than or equal to the design resistance, R_d

$$E_d \leq R_d$$

and for serviceability limit states that the design effect of the actions, E_d, must be less than or equal to the limiting design value of the relevant serviceability criterion, C_d

$$E_d \leq C_d$$

Geotechnical design differs from design in other structural materials in that both the design actions and the design resistances are functions of the actions, material properties and dimensions of the problem. For example, when assessing sliding in a retaining wall design the horizontal forces due to the earth (design effect of the actions) are derived from the material properties of the soil and the dimensions of the problem; similarly, the resistance to sliding (design resistance) is also derived from the actions, material properties and the dimensions. Thus, in this regard, geotechnical calculations have a greater level of complexity than design in other structural materials where actions are not normally a function of the material properties and resistances not a function of the actions.

Limit states should be verified by calculation, prescriptive measures, experimental models and load tests, an observational method, or a combination of these approaches.

Design by calculation involves a number of elements. These include three basic variables: actions (e.g. the weight of soil and rock, earth and water pressures, traffic loads, etc.), material properties (e.g. the density and strength of soils, rocks and other materials) and geometrical data (e.g. foundation dimensions, excavation depths, eccentricity of loading, etc.).

The basic variables are entered into calculation models, which may include simplifications but are required to 'err on the side of safety'. These models may be analytical (e.g. bearing capacity theory), semi-empirical (e.g. the alpha method of pile design) or numerical (e.g. finite element analysis).

Figure 7.4 indicates where the partial factors used in design by calculation may be applied. Two 'sets' of partial factors γ_F (A1 and A2) are provided for actions, two sets of partial factors γ_M (M1 and M2) for material properties and four sets of partial factors γ_R (R1, R2, R3 and R4) are provided for resistances.

Eurocode 7 presents three different design approaches for carrying out ultimate limit state analysis for the GEO and STR limit states. These approaches apply the partial factor sets in different combinations in order to provide reliability in the design. The method of applying the partial factors reflects the differing opinions regarding geotechnical design held across the European Union. In essence, Design Approach 1 provides reliability by applying different partial factor sets to two variables in two separate calculations ('Combinations' 1 and 2), whereas Design Approaches 2 and 3 apply factor sets to two variables simultaneously, in a single calculation as outlined in Table 7.4.

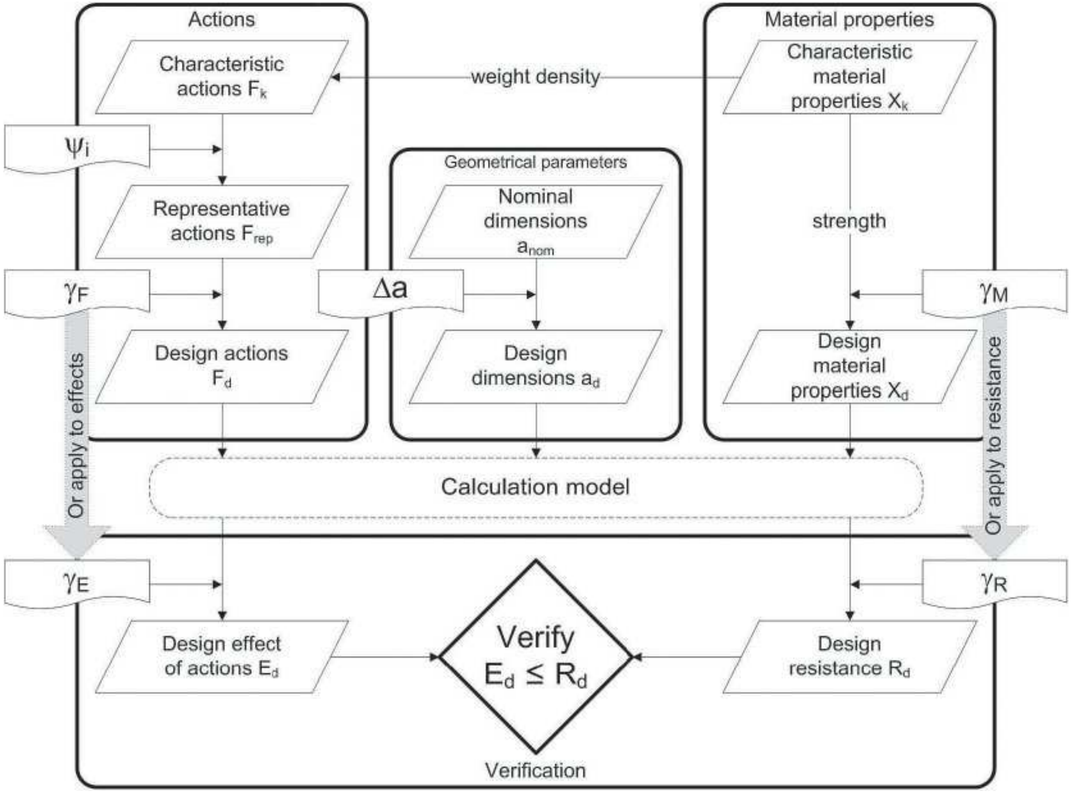


Figure 7.4. Overview of verification of strength (Bond and Harris 2008)

Table 7.4. Design approaches

Structure	**Main variable that gets factored in Design Approach**			
	1		**2**	**3**
	Combination 1	**Combination 2**		
General	Actions	Material properties	Actions (or effects) and resistances	Structural actions (or effects) and material properties
Slopes			Effects of actions and resistances	Structural effects of actions and material properties
Piles and anchorages		Resistances	Actions (or effects) and resistances	Structural actions (or effects) and material properties

In the UK, Design Approach 1 is being adopted, requiring two calculations to be carried out: Combination 1 where actions are factored and material properties are left unchanged and Combination 2 where material properties are factored and only variable unfavourable actions are factored.

For example, consider a simple rectangular footing of length 2.5 m, breadth 1.5 m and thickness 0.5 m which is founded at a depth of 0.5 m in dry sand. The sand has a weight density of 18 kN/m^3 with a characteristic angle of friction, ϕ'_k, of 35° and the footing is subject to a permanent vertical action of 800 kN and variable action of 450 kN.

For Combination 1 the permanent actions are increased by the partial factor $\gamma_G = 1.35$ and the variable actions are increased by the partial factor $\gamma_Q = 1.5$. Thus the design vertical action is the weight of the footing plus the loads acting on it multiplied by the relevant partial factor, giving a design effect of the actions $E_d = 1818.3$ kN. The design resistances are calculated from the design angle of friction, for Combination 1, using partial factor $\gamma_\phi = 1.0$, therefore $\phi'_d = 35°$. Using standard bearing capacity theory as given in Annex D of EN 1997-1 gives a design resistance $R_d = 3388 1$ kN; thus, for Combination 1, $R_d >> E_d$.

For Combination 2 the permanent actions are increased by the partial factor $\gamma_G = 1.0$ and the variable actions are increased by the partial factor, $\gamma_Q = 1.3$. Thus the design vertical action is the weight of the footing plus the loads acting on it multiplied by the relevant partial factor, giving a design effect of the actions $E_d = 1431.9$ kN. The design resistances are calculated from the design angle of friction, for Combination 2, using partial factor $\gamma_\phi = 1.25$ therefore $\phi'_d = 29.3°$. Using standard bearing capacity theory as given in Annex D of EN 1997-1 gives a design resistance $R_d = 1479$ kN. Thus for Combination 2, R_d is only marginally greater than E_d and Combination 2 governs the geotechnical design. For a full discussion of this example and similar problems refer to Bond and Harris (2008) [2].

'Prescriptive measures' are a combination of conservative design rules and strict control of execution that, if adopted, avoid the occurrence of limit states. The design rules, which often follow local convention, are commonly set by local or national authorities, via building regulations, government design manuals, and other such documents. These design rules may be given in a country's national annex to EN 1997-1. Design by prescriptive measures may be more appropriate than design by calculation, especially when there is 'comparable experience', i.e. documented (or other clearly established information) in similar ground conditions, involving similar structures – suggesting similar geotechnical behaviour.

Eurocode 7 acknowledges the role of large- and small-scale model tests in justifying the design of geotechnical structures by calculation, prescriptive measures or observation. However, apart from requiring time and scale effects to be considered and differences between the test and real construction to be allowed for, EN 1997-1 provides very little guidance on design by testing.

In a similar way to design by testing, Eurocode 7 acknowledges the role of the observational method in the design and construction of geotechnical structures, but provides little guidance on how to implement it. Certain actions must be taken: establish limits of behaviour, assess the range of possible behaviour, devise a plan of monitoring, devise a contingency plan and adopt it if behaviour goes outside acceptable limits.

Supervision, monitoring and maintenance

Eurocode 7 has specific requirements to ensure the quality and safety of a structure including: supervision of the construction process and workmanship, monitoring the performance of the structure, checking ground conditions, checking construction and maintenance. EN 1997-1 qualifies this requirement by saying these tasks should be undertaken 'as appropriate'. Thus, if construction does not need supervising, or the structure does not need monitoring or maintaining, then the design could explicitly rule out the need for them.

Supervision involves checking design assumptions are valid, identifying differences between actual and assumed ground conditions, and checking construction is carried out according to the design. A plan of supervision should be prepared, stating acceptable limits for any results obtained, and indicating the type, quality and frequency of supervision. The amount and degree of inspection and control and the checking of ground conditions and construction all depend on the geotechnical category (GC) in which the structure is placed. The fact that EN 1997-1 relates its requirements for supervision to the geotechnical categories may make their adoption difficult to avoid.

Monitoring involves measurements of ground deformations, actions, contact pressures, and such like and an evaluation of the performance of the structure. For structures placed in Geotechnical Category 1, such evaluation is simple, qualitative and based largely on inspection. For Category 2 structures, the movement of selected points on the structure should be measured. Finally, for Category 3 structures, monitoring also involves analysis of the construction sequence.

If an alternative method of assessing geotechnical risk is used, then the amount of inspection, monitoring and control should be selected appropriately for the risk involved.

Maintenance of a structure ensures its safety and serviceability and the requirements for it should be specified to the owner or client. These requirements include identifying those parts of the structure that require regular inspection, warning about work that should not be undertaken without prior design review and indicating the frequency of inspection.

Although EN 1997-1 imposes specific requirements for supervision, monitoring and maintenance, it gives few practical recommendations as to the actions that need to be taken to meet those requirements. This is demonstrated by the small number of paragraphs on this subject in the whole of Part 1 (thirty-three), broken down as follows: clause 5 Fill, dewatering, etc., eight paragraphs; clause 6 Spread foundations, two paragraphs; clause 7 Pile foundations, eight paragraphs; clause 8 Anchorages, eight paragraphs; clause 9 Retaining structures, none; clause 11 Overall stability, two paragraphs; and clause 12 Embankments, five paragraphs.

EN 1997-2 *Ground investigation and testing*

The values provided in any partial factor system are swamped by the potential variability in geotechnical parameters both laterally and with depth. It is thus critical to any design that suitably conservative models are selected on the basis of the information available. EN 1997-2 provides detailed guidance on how to derive geotechnical parameters both directly from field tests and from laboratory work. It also emphasizes the importance of adequate site investigation providing guidance on the minimum requirements for the numbers and depths of investigation points as well as the associated testing.

Further, it details the reporting requirements for a design to conform to Eurocode 7 which encapsulate much of what is current good practice but seeks to bring greater coordination between the structural and geotechnical engineer. This is accomplished through a series of reports culminating in the Geotechnical Design Report as summarized in Figure 7.5.

The Geotechnical Design Report is a key feature of a Eurocode 7 design, bringing together all the factual data and interpretation leading to the selection of an appropriate ground model and characteristic parameters as shown

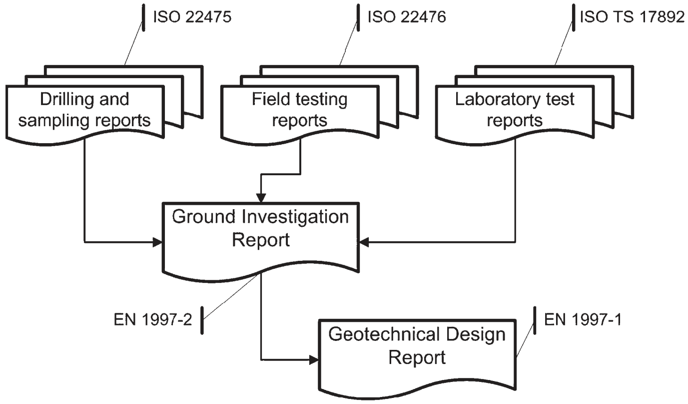


Figure 7.5. Summary of geotechnical reporting to Eurocode 7 (Bond and Harris 2008)

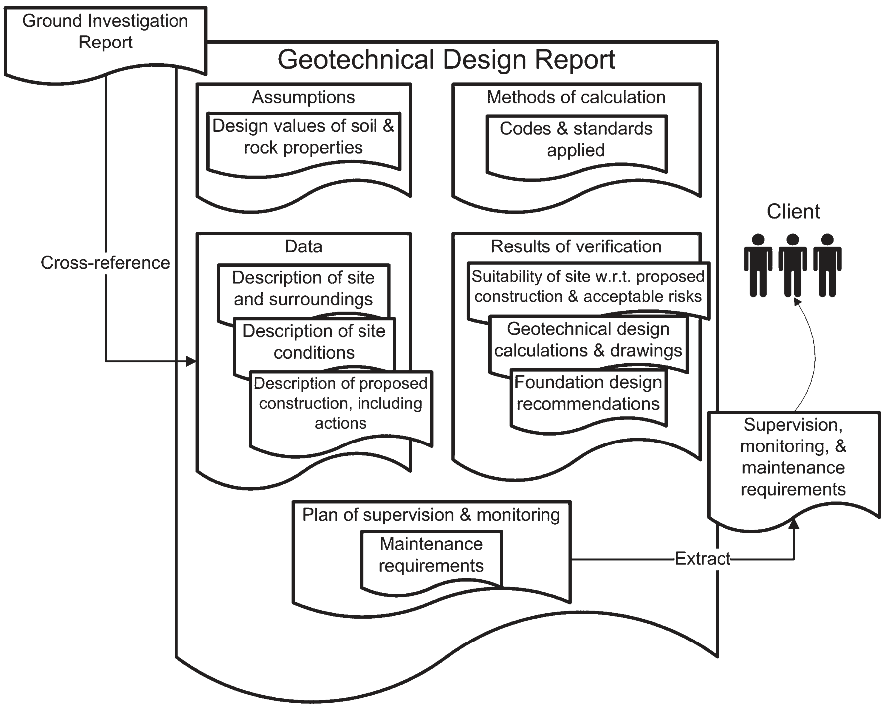


Figure 7.6. Contents of the geotechnical design report (Bond and Harris 2008)

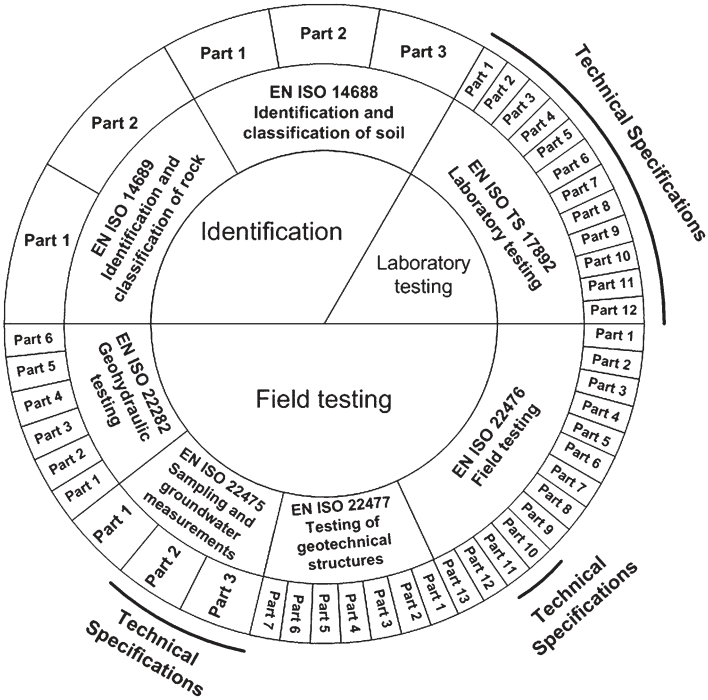


Figure 7.7. Complementary geotechnical investigation and testing standards (Bond and Harris 2008)

in Figure 7.6. The presentation of a traditional interpretative report by a site investigation contractor may no longer be appropriate as Eurocode 7 requires differing characteristic parameters for different limit state calculations. In addition the design is crucially dependent on a knowledge of the permanent and variable actions, which are often unknown at the investigation stage.

EN 1997-2 itself does not contain all the detail necessary to perform in situ and laboratory tests so it refers extensively to a new suite of international and European standards, prepared jointly by ISO technical committee TC 182 and CEN TC 341, as summarized in Figure 7.7.

Each of the standards within each group is divided into a number of parts, as shown in the outer ring of Figure 7.7. The entire suite comprises nearly fifty standards or specifications.

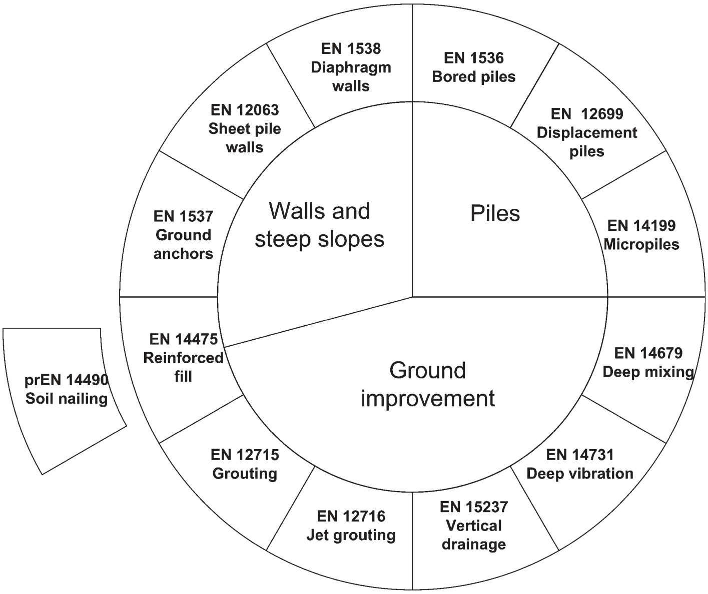


Figure 7.8. Overview of the geotechnical execution standards (Bond and Harris 2008)

It also does not contain information on how to carry out geotechnical processes and refers to a further series of standards covering execution as summarized in Figure 7.8.

Concluding remarks

The introduction of the Eurocodes will challenge current design processes and introduce greater rigour into geotechnical design. However, there will be considerable concern that experience built up over many years will be lost or difficult to apply in this new regime.

EN 1997-1 and 2 are relatively short documents and do not contain the detailed guidance found in such current British Standards as BS 5930, BS 8002 and BS 8004 or other related publications such as CIRIA C580. However, the new codes refer to a large number of other documents that will provide the detail. Further work is currently underway to modify the existing British

Standards so that they do not conflict with the Eurocodes, ensuring that they can continue to be used.

As experience is gained in using the codes it is hoped that a unified approach to GEO and STR may be developed avoiding the complications of different design approaches and enabling greater transparency across the European Union. Further it is likely that the partial factors themselves will be modified to ensure economical design.

References

[1] Driscoll R., Powell J. and Scott P. *A designer's simple guide to BS EN 1997*, London: Dept for Communities and Local Government, 2007

[2] Bond A. J. and Harris A. J. *Decoding Eurocode 7*, Taylor and Francis, 2008

Eurocode 8: Design of structures for earthquake resistance

Edmund Booth, Consulting Engineer

Introduction

The six Parts of EN 1998 form a comprehensive set of requirements that provide a unified approach to the seismic design of structures and their foundations. The stated purpose of EN 1998 is to ensure that in the event of earthquakes:

- human lives are protected;
- damage is limited; and
- structures important for civil protection remain operational.

EN 1998 covers not only building structures, but also bridges and other facilities such as chimneys, towers, tanks and pipelines (both buried and above ground). Dams, offshore structures, nuclear power stations and long span suspension bridges are however specifically excluded from its scope.

Table 8.1 shows the different parts of EN 1998 and the years of publication of the UK national annexes, which are available for all Parts except Part 3. Part 1 (EN 1998-1) contains general material applicable to all types of structure covered by EN 1998, including the definition of seismic actions. Part 1 also contains the main rules specific to the design of building superstructures in concrete, steel, steel–concrete composite, timber and masonry. The use of base isolation bearings to provide seismic protection is also covered by Part 1. Part 5 covers geotechnical matters, including the design of foundations, and therefore, like Part 1, applies to all ground supported structures. Parts 2, 4 and 5 provide additional rules for specific structural types other than buildings. Part 3 of EN 1998 deals with the assessment and retrofit of existing buildings, which is an important issue for seismic regions of the world, where

Table 8.1. The six Parts of EN 1998 – *Design of structures for earthquake resistance*

Part of Eurocode 8	Year of issue of UK national annex
EN 1998-1: *General rules, seismic actions and rules for buildings:* Section 1: General Section 2: Performance requirements and acceptance criteria Section 3: Ground conditions and seismic action Section 4: Design of buildings Sections 5–9: Specific rules for buildings in concrete, steel, composite steel–concrete, timber and masonry Section 10: Base isolation	
EN 1998-2: *Bridges*	2009
EN 1998-3: *Assessment and retrofitting of buildings*	None proposed
EN 1998-4: *Tanks, silos and pipelines*	2008
EN 1998-5: *Foundations, retaining structures and geotechnical aspects*	2008
EN 1998-6: *Towers, masts and chimneys*	2008

there are many buildings for which construction predated modern seismic design codes or where the general quality of historic infrastructure is seismically inadequate. No UK national annex is proposed for Part 3 because it was considered insufficient use would be made of it for structures in the UK which lay within the scope of EN 1998.

As can be seen from Table 8.1, EN 1998 provides a unified approach to the seismic design of a very wide range of structural types and construction materials. It covers the selection of design ground motions, seismic analysis, special seismic detailing requirements and geotechnical issues such as design of retaining walls and assessment of the liquefaction potential of soils. EN 1998 is of course fully integrated with the rest of the Eurocode suite, to which reference is needed for non-seismic aspects of design. The comprehensive scope of EN 1998, and its ability to form part of a uniform basis for all aspects of design, is an important feature of the code which will be of benefit to UK designers. The clear and rational basis for its provisions and the advanced nature of many of its procedures will also assist designers.

Use of EN 1998 for design of structures in the United Kingdom

Requirements for seismic design in the UK

Seismic design has not previously been required for the great majority of UK structures and the introduction of EN 1998 does not change this. All the UK National Forewords to EN 1998 state:

> *'There are generally no requirements in the UK to consider seismic loading, and the whole of the UK may be considered an area of very low seismicity in which the provisions of EN 1998 need not apply. However, certain types of structure, by reason of their function, location or form, may warrant an explicit consideration of seismic actions. Further guidance on the circumstances where an explicit seismic design should be considered is provided in PD 6698:2009.'*

The UK national annexes to EN 1998 advise that an assessment of the need for seismic loading should only be carried out for consequence category CC3 structures. These are defined in EN 1990 Table B.1 as structures where failure would lead to high consequences for human life, or considerable environmental, social or economic impact. However, for many category CC3 structures, the recommended assessment is likely to conclude that an explicit seismic design is not required. The need for seismic design will depend upon the consequences of seismically induced failure, the level of seismicity at the site in relation to the UK average and the presence or absence of structural features known to be unfavourable for seismic resistance. Semi-quantitative guidance on these choices is provided in PD 6698:2009 (see next section), but it is recognized that much will depend on judgement.

PD 6698:2009 advises that structures in consequence categories CC1 and CC2 are 'unlikely to warrant seismic design, provided they are adequately designed for non-seismic design conditions'. These are structures where the consequences of loss are low or medium, and their seismic resistance can be considered as adequately covered by the robustness provisions of the non-seismic parts of the Eurocodes.

Published Document PD 6698:2009 – Recommendations for the design of structures for earthquake resistance to BS EN 1998

PD 6698:2009 is a background paper that gives non-contradictory, complementary information on the use of EN 1998 for the seismic design of UK structures. Although published by BSI, it does not have the status of a British Standard. PD 6698 contains a seismic hazard zoning map of the UK which the UK national annexes to EN 1998 permit as one option for obtaining ground motions for use in seismic design. An alternative option given by the UK national annex is to undertake a site specific hazard analysis; the UK national annex to Part 4 requires such analysis for the design of petrochemical facilities associated with large risks to the population or the environment. PD 6698 also provides advice on the circumstances in which seismic design may be warranted for buildings, bridges, tanks, pipelines, chimneys and towers in consequence category CC3.

Seismic design of nuclear facilities, dams and offshore structures

Nuclear power plants are a prime example of 'high consequence of failure' structures, and have been designed seismically in the UK since the 1980s; major dams have also been subject to seismic assessment for many years as have offshore structures. However, these types of structures are specifically excluded from the scope of EN 1998 because there are particular aspects of their regulatory and detailed design performance that are not covered. Notwithstanding this, many features of EN 1998 are still relevant, and it is likely that EN 1998 will influence the practice of UK designers undertaking nuclear dam and offshore design work in the future.

Seismic design of bridges and petrochemical facilities

Certain petrochemical facilities, such as liquid natural gas (LNG) tanks and high pressure gas pipelines, and important bridges are examples of other types of UK structures where seismic design has sometimes been carried out in the past, and EN 1998 covers seismic aspects of their design.

EN 1998-2 covers the seismic design of bridges. PD 6698:2009 gives advice on the circumstances where seismic design needs to be considered for important

UK bridges, and identifies the sections of EN 1998-2 which are applicable in the UK. Many of the provisions of EN 1998-2 do not apply to bridge design in low seismicity regions such as the UK; the main requirement is to carry out a seismic analysis and to check that the forces and deflections that this predicts are within the strength and deformation capacity of the bridge. The special seismic detailing provisions in EN 1998-2 are not generally required, as they would be in an area of high seismicity.

EN 1998-4 covers the seismic design of tanks and pipelines. Tanks are the only type of structures where seismic design was previously covered by British Standards. There are two current relevant British Standards. BS EN 1473:2007 covers onshore LNG installations; it refers to EN 1998-4 for the seismic design of tanks. BS EN 14015:2004 covers certain types of flat bottomed tanks for storage of liquids at ambient temperature and above; this currently contains a seismic design procedure based on US practice of some time ago, but it is understood that a revision will be published shortly which replace this advice with a requirement to use EN 1998-4. PD 6698:2009 currently advises that EN 1998-4 should be used in place of the seismic provisions of BS EN 14015:2004.

Seismic design of buildings

Generally speaking, the only UK buildings which have previously been seismically designed were those with a safety related function, for example in the nuclear industry. The introduction of EN 1998, together with the UK national annexes and PD 6698:2009, opens the possibility that a limited range of other types of buildings may warrant seismic design, although as noted above, the UK national annexes state ‘that the whole of the UK may be considered an area of very low seismicity in which the provisions of EN 1998 need not apply’ and so seismic design is not mandatory. An example of a consequence category CC3 building which might warrant seismic design is a hospital in a part of the UK shown by the seismic zoning map of PD 6698:2009 to have higher than average seismicity, and which is either sited on soft soils (which tend to amplify earthquake motions) or has some structural feature unfavourable to seismic resistance such as a weak or ‘soft’ ground storey.

If seismic design were carried out, it would be similar to that described in the section for bridges as follows. That is, the main requirement would be to carry out a seismic analysis and to check that the forces this predicts are within the strength capacity of the building structure. The special seismic detailing provisions of EN 1998-1 would not generally be required.

Use of EN 1998 in areas of moderate to high seismicity worldwide

Introduction

The previous section applies to the special and rather unusual situations where EN 1998 is used for the seismic design of structures in the UK. It is expected that much more use will be made of EN 1998 by UK designers for seismic design of structures outside of the UK than within in it, for parts of the world where earthquakes provide a much more immediate threat. Within those countries adopting the Eurocodes, designs would of course need to make use of the relevant national annexes, rather than the UK national annex; these provide the seismic zoning maps needed for establishing design seismic loading and also the values of the Nationally Determined Parameters (NDPs). Advice on the application of EN 1998 to countries where a national annex does not exist is provided by the Institution of Structural Engineers/ AFPS Manual on EN 1998 [1].

Many UK engineers in the past have used US seismic codes for seismic designs internationally. EN 1998 has a number of features which are rather different from US practice. Broadly, the provisions of EN 1998 can be characterized as less empirical and more related to fundamental physics than the US counterparts, and in some places it may seem closer to an academic textbook than to the type of code provisions that UK engineers are used to. The common approach to a wide variety of structural types is an advantage over US provisions, as is the much greater provision of advice on geotechnical matters, given in EN 1998-5. The more rapid revision cycle in the US means that certain parts of current US codes can be regarded as more up to date; the EN 1998 Manual [1] gives advice on some circumstances where the provisions of EN 1998 may need upgrading. Generally, EN 1998 requires considerably greater lateral strength than do US codes for equivalent buildings.

Subsequent paragraphs give a brief outline of some of the principal features of EN 1998.

Analysis methods

In common with most current seismic codes, ductility modified response spectrum analysis is the main type of analysis envisaged for buildings by EN 1998. The 'behaviour factor' q in EN 1998 is broadly equivalent to the 'response modification factor' R of the US International Building Code (IBC). The behaviour factor allows for the reduction in lateral strength requirement in structures that are designed to yield in a ductile manner during a major earthquake; the greater the degree of ductility that is available in the structure, the larger the q factor. Static pushover analysis, using displacement-based methods of analysis, is a more recent and precise way of allowing for post-elastic response. It is also presented in EN 1998, but not fully developed for new buildings in Part 1. However, complete methods are provided for bridges in Part 2 and for retrofitting buildings in Part 3. Time history and (briefly) power spectrum methods of analysis are also referred to.

An outline of some basic principles in EN 1998 for building design

Design for different levels of ductility

A well-established principle in earthquake engineering is the trade-off between strength and ductility; if collapse prevention and life safety preservation is the only concern, then for a given intensity of earthquake loading, a lower strength building can be designed, providing that the detailing ensures proportionately more ductility.

EN 1998 allows the seismic designer to make such a trade-off by specifying two main classes of ductility provision, namely Ductility Class High (DCH) and Ductility Class Medium (DCM). The lateral strength requirement in DCM structures is 25% to 50% greater than in DCH structures. There are quite stringent seismic design and detailing requirements for both DCH and DCM structures, but they are more complex for DCH than DCM, particularly for concrete structures. Note that in US practice, 'special' ductility provisions are required for buildings in areas of high seismicity; generally, these 'special' provisions probably lie somewhere between those of DCH and DCM.

A third, low-ductility class exists, Ductility Class Low (DCL), but this is not permitted for use in areas of moderate or high seismicity. DCL structures must be designed for essentially elastic response, using a q factor not exceeding 1.5 (or 2 for some steel structures). However, after providing sufficient lateral strength to resist the seismic forces implied by $q \leq 1.5$ (or $q \leq 2$ in steel structures), almost no further specific seismic design and detailing measures are required, and design may proceed using the appropriate non-seismic Eurocodes.

The reason for prohibiting DCL structures in areas of moderate or high seismicity is that they may fail in a brittle manner if loaded beyond their design strength. Since both the level of seismicity and seismic response are subject to great uncertainties, a high level of ductility is a good insurance policy; ductile structures, while they will be damaged by greater than expected earthquake motions, are much less likely to collapse and kill or injure their occupants than brittle structures. Capacity design measures (see section below) are an essential part of ensuring that this reserve of ductility is present.

Primary and secondary elements

Primary elements are those which contribute to the seismic resistance of the structure. Some structural elements can however be designated as 'secondary' elements, which are taken as resisting gravity loads only. Their contribution to seismic resistance must be neglected. These elements must be shown to be capable of maintaining their ability to support the gravity loads under the maximum deflections occurring during the design earthquake. This may be done by showing that the actions (moments, shears, axial forces) that develop in them under the enforced compatible seismic deformations for the whole structure (determined by analysis) do not exceed their design strength, as calculated using Eurocode 2 or Eurocode 3. Otherwise no further seismic design or detailing requirements are required.

Essentially, secondary elements are a type of low ductility element, which (unlike DCL elements) are not restricted to areas of low seismicity.

Figure 8.1 shows two framing arrangements. In the grid frame, all the columns are primary elements, but in the perimeter frame, the internal columns are secondary elements. Thus, the perimeter frame is considered as the primary seismic resisting element, and is designed for high ductility while the internal members are considered secondary. This gives considerable architectural freedom for the layout of the internal spaces; the column spacing can be much greater than would be efficient in a moment resisting frame, while close spaced columns on the perimeter represents much less obstruction.

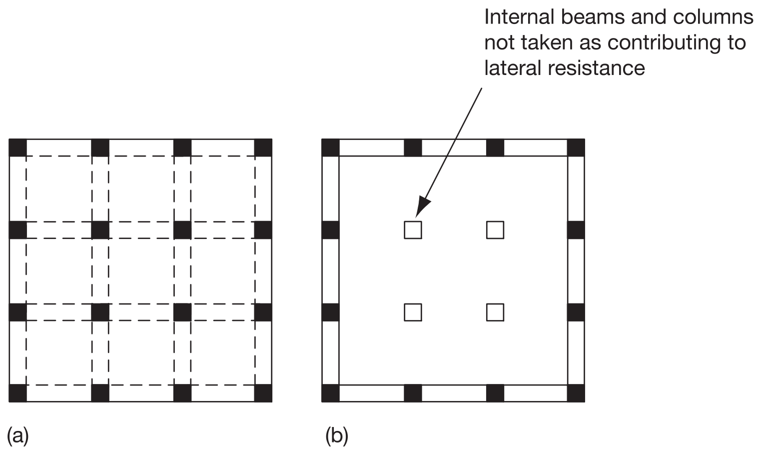


Figure 8.1. Sectional plan on grid and perimeter frames. (a) Grid frame. (b) Perimeter frame

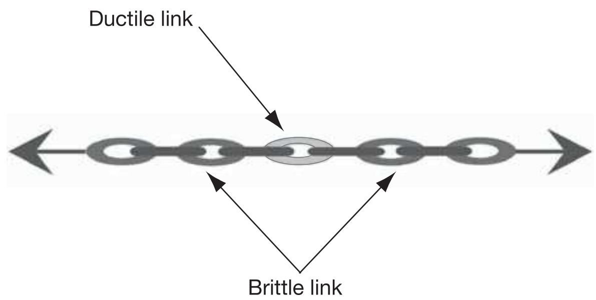


Figure 8.2. Concept of capacity design

Capacity design

DCH and DCM structures must be checked by capacity design procedures to ensure that ductile yielding occurs before brittle failure. The principle is to ensure that the ductile regions yield first, thereby protecting the brittle regions from reaching their capacity. This is done by ensuring that brittle elements are designed to withstand a load which exceeds that induced by the yielding strength of the ductile links. The ductile links then act as a structural fuse, ensuring that the brittle links do not reach their capacity. This concept, developed by Professors Park & Paulay in Christchurch New Zealand in the 1970s, is illustrated schematically by the 'ductile chain' in Figure 8.2.

Table 8.2. Recommended definitions of seismic zones in EN 1998

Zone	Peak ground acceleration (PGA) for 475 year return period		Seismic design requirements
	Rock	Soil	
High to moderate	> 8 %g	> 10 %g	Full seismic detailing to DCH or DCM
Low	8 %g ≥ PGA > 4 %g	10 %g ≥ PGA > 5 %g	DCL; $q \leq 1.5$ ($q \leq 2$ for steel) no seismic detailing
Very low	≤ 4 %g	≤ 5 %g	No seismic design required

Ensuring that columns are stronger than beams in moment frames, and that concrete frame members are stronger in shear than in flexure are two important examples of capacity design. The principle is simple, though the rules in EN 1998, particularly for concrete structures, can be quite complex. The capacity design principle is applied more widely than is the case for US seismic codes; for example, foundations must generally be checked to capacity design principles to ensure they can develop the yielding strength of the superstructures they support, which is not required by US codes.

Design for different levels of seismicity

Three bands of seismicity are recognized by EN 1998, as follows.

- high to moderate;
- low;
- very low.

In high to moderate seismic zones, building structures should be designed with ‘high’ (DCH) or ‘medium’ (DCM) ductility; this involves full seismic detailing and capacity design procedures. In low seismicity zones, structures may be designed as ‘low’ ductility and special seismic detailing and capacity design requirements are waived. Seismic design is waived altogether in very low seismicity zones.

The recommended definitions for the three zones given in EN 1998 are given in Table 8.2, although they can be modified for a particular country by its national standards authority. As noted previously, the UK national annexes to EN 1998 define the whole of the UK as being a zone of very low seismicity.

Guidance material on EN 1998

The *Designers' Guide to EN 1998-1 and EN 1998-5* [2] gives background information and guidance on many aspects of EN 1998. It is one of a series of designers' guides to the Eurocode suite.

A manual on EN 1998 [1] has been prepared by the Institution of Structural Engineers and the Association Française du Genie Parasismique (AFPS – which is the French counterpart of the UK earthquake society SECED). It is intended as a stand-alone source for the requirements of EN 1998 for the majority of straightforward steel and concrete buildings, and in addition provides extensive guidance and design aids beyond those provided in EN 1998 itself. The manual gives the values of NDP recommended by EN 1998, as well as those given in the UK and French national annexes. Seismic zoning maps of the UK and France are provided.

A textbook edited by Elghazouli [3] based on recent courses presented by SECED at Imperial College gives further information on structural and geotechnical aspects of EN 1998, and includes worked examples for the foundation and structural design of an example building, constructed in both concrete and steel.

An article in the Structural Engineer [4] provides a briefing on EN 1998.

A textbook by Booth and Key [5] makes extensive reference to the provisions of EN 1998.

References

[1] *Manual for the seismic design of steel and concrete buildings to EN 1998*, Institution of Structural Engineers, London: 2009

[2] Fardis M., Carvalho E., Elnashai A., Faccioli E., Pinto P. and Plumier A. *Designers' guide to EN 1998-1 and 1998-5. EN 1998: Design Provisions for Earthquake Resistant Structures*, Thomas Telford, London, 2005

[3] Elghazouli A. (editor) *Seismic design of buildings to EN 1998*, Spon Press, London, 2009

[4] Standing Committee on Implementation of Eurocodes. Eurocode 8 explained, *The Structural Engineer*, Vol. 87, No. 12, 16 June 2009, p15

[5] Booth E. and Key D. *Earthquake design practice for buildings*, Thomas Telford, London, 2006

Eurocode 9: Design of aluminium structures

Phil Tindall, UK Technical Director (Bridges), Hyder Consulting

There are five Parts to Eurocode 9 covering the design of aluminium structures:

- BS EN 1999-1-1:2007 *General structural rules*
- BS EN 1999-1-2:2007 *Structural fire design*
- BS EN 1999-1-3:2007 *Structures susceptible to fatigue*
- BS EN 1999-1-4:2007 *Cold-formed structural sheeting*
- BS EN 1999-1-5:2007 *Shell structures*

The rules given in Parts 1-1 and 1-3 for structural and fatigue design are largely equivalent to those given in BS 8118-1:1991, *Structural use of aluminium, Part 1: Code of practice for design*. The material in the other three Parts was not previously covered by a British Standard in any great detail.

Eurocode 9, in common with other Eurocodes, makes considerable cross-reference to other European Standards. In particular, it is based on the principles contained in Eurocode 0 (*Basis of structural design*) and refers to Eurocode 1 for loading.

The design rules are inextricably linked to the rules for execution (the term that Eurocodes use for fabrication and erection) given in BS EN 1090-3: *Execution of steel structures and aluminium structures, Part 3: Technical requirements for aluminium structures*. BS EN 1090-3 will replace BS 8118-2:1991, *Structural use of aluminium, Part 2: Specification for materials, workmanship and protection*.

Each Part of Eurocode 9 has an accompanying national annex which gives UK-specific partial factors and choices. The use of the UK national annexes is a prerequisite for the use of Eurocode 9.

BSI is issuing two documents to assist UK designers using Eurocode 9 in the UK. These are:

- PD 6702-1, *Structural use of aluminium, Part 1: Recommendations for the design of aluminium structures to BS EN 1999*
- PD 6705-3, *Structural use of aluminium, Part 3: Recommendations for the execution of aluminium structures to BS EN 1090-3*

These documents give important guidance on matters where choice is given in Eurocode 9 or BS EN 1090-3, together with additional background data referred to in the national annexes.

BS EN 1999-1-1:2007 *General structural rules*

Most of the rules in Part 1-1 of Eurocode 9 are similar to those in BS 8118 and are generally based on the same structural principles.

The rules are, however, more extensive and allow greater refinement of the design. This can lead to more economical structures when dealing with complex or very slender sections, albeit at the expense of more extensive and complex design checking. Comparative exercises between Eurocode 9 and BS 8118 have shown that the difference in allowable loads is small for static design of typical members and details. Presently many designers use commercial software or in-house spreadsheets for code compliance checking; consequently more complex checks are not necessarily an issue. In common with other Eurocodes, the additional clauses and the need to reference other standards increases the required design effort.

In the conversion from prENV 1999 (published in the UK in 2000 as a Draft for Development) considerable efforts were made to align the format and content of Eurocode 9 with Eurocode 3, on the basis that designers familiar with steel would more easily follow the aluminium code. This does, however, ignore the fact that aluminium is a different material that has its own properties that need to be exploited and worked with differently to steel if an economic structure is to be achieved.

Several of the notable aspects that are different to the design rules of BS 8118 are listed as follows.

- Eurocode 9 (clause 1.8) requires that a specification is prepared for execution of the work, whereas BS 8118-2 acted as the specification. There are various clauses in both Eurocode 9 and EN 1090-3 where there are alternatives and/or items that have to be specified by the designer. PD 6702-1 and PD 6705-3 give guidance and recommendations to help the designer to make appropriate choices.

- The concept of reliability differentiation is introduced. Varying quality requirements, and varying degrees of assurance that the work meets the specified quality level, are implemented by the use of service categories and execution classes. Again, PD 6702-1 and PD 6705-3 give guidance and recommendations to help the designer make appropriate choices. Note that the service categories given in these documents differ from those used in EN 1090-3. (See further commentary on this in the section on BS EN 1999-1-3 below.)
- The various partial safety factors in Eurocode 9 and Eurocode 0 have different values to those in BS 8118 – however the final combination of all of the factors on the loading and resistance sides of the equation give similar results. The use of common loads and load factors for application across the whole range of structural materials is welcome.
- Eurocode 9 introduces an additional class when classifying cross sections for buckling resistance. The additional class is for sections that can form a plastic hinge with the rotation capacity required for plastic analysis.
- Eurocode 9 generally gives higher weld strengths and allows the designer to calculate differing weld metal strengths to match the weld metal specified, whereas BS 8118 used a lower-bound figure. The Beta formula used for calculating stresses in a fillet weld is slightly different to that used in BS 8118, and this combined with the higher weld metal strengths prompted the UK national annex to specify a higher value for the partial safety factor (Gamma m) for welded joints.
- Eurocode 9 gives simple design rules for the structural use of castings. In general, these are only applicable for static applications when impacts and fatigue consideration are not relevant. BS 8118 did not give any design rules for castings.

Eurocode 9 includes several informative annexes that introduce additional guidance or methods that were not covered explicitly in BS 8118. These include material on analytical models for stress–strain relationship, behaviour of materials beyond the elastic limit, plastic hinge method for continuous beams, shear lag effects and classification of joints.

Other informative annexes include and sometimes expand on information that was included in BS 8118 such as material selection, corrosion and surface protection, and properties of cross-sections.

The national annex to this Part of Eurocode 9 refers to PD 6702-1 and PD 6705-3 for additional guidance as noted above and for guidance on deflections and vibration limits in buildings. All of the informative annexes in this Part of Eurocode 9 are permitted and retain an informative status.

BS EN 1999-1-2:2007 *Structural fire design*

Structural fire design of aluminium structures was not previously covered by British Standards at all. This Part of Eurocode 9 gives comprehensive rules for determining the fire resistance of aluminium members in structures. Contrary to popular opinion, aluminium is classified as non-combustible and this section is a welcome addition to the suite of standards. Previously, knowledge of the subject was confined to a small group of experts.

The theory of the fire safety of structures in aluminium alloys is governed by the same principles and methods as those used for steel structures. Whilst most aluminium alloys start to lose some strength when held at temperatures above 100 °C and have lost a significant proportion by 350°C, applications that need insulation for extended fire resistance are generally similar to those for steel structures.

Rules are given for structures that are unprotected, that are insulated by fire protection material or are protected by heat screens.

The national annex to this Part of Eurocode 9 does not change any of the recommended values. All of the informative annexes in this Part of Eurocode 9 are permitted and retain an informative status.

BS EN 1999-1-3:2007 *Structures susceptible to fatigue*

Eurocode 9 gives methods for calculation of fatigue life based on 'safe life' principles or on a damage-tolerant approach, whereas BS 8118 only gave guidance and methods for fatigue design based on 'safe life' principles.

The Eurocode also gives greater detail on items such as methods of structural analysis and stress concentrations applicable for fatigue design.

The methodology used by Eurocode 9 for the safe life approach is generally similar to that used in BS 8118. The basis of the design methodology is given both in the main text and Annex A of Eurocode 9 Part 1-3. The detail category tables necessary to calculate the fatigue life are given in informative Annex J. The BSI mirror committee responsible for Eurocode 9 considered that some of the fatigue detail categories in Annex J could be subject to misinterpretation, or could give fatigue safe lives only achievable with unrealistic expectations regarding internal defects. The UK recommendation is not to use the detailed categories contained in the informative annex. Alternative detail category tables are therefore given in PD 6702-1, *Structural use of aluminium, Part 1: Recommendations for the design of aluminium structures to BS EN 1999*.

Realization of the predicted fatigue lives is dependent on achieving certain quality levels. However, it is recognized that there could be an economic penalty for over-specifying quality requirements for areas subject to static loading or low levels of cyclic stress. The alternative fatigue detail category tables are therefore associated with a series of quantified service categories that can be used with the recommendations in PD 6705-3, *Structural use of aluminium, Part 3: Recommendations for the execution of aluminium structures to BS EN 1090-3* to specify appropriate inspection regimes and acceptance criteria during execution. The alternative detailed category information is based on data previously issued in prENV 1999-2, published in the UK in 2000 as a Draft for Development.

The Eurocode also allows a damage-tolerant approach to fatigue design, i.e. some cracking is allowed to occur in service, provided that there is stable, predictable crack growth and that there is a suitable inspection regime in place. The UK recommendation is that design should be based on safe life principles whenever possible, but recognizes that there may be situations where achievement of minimum weight is a high priority and in circumstances where the necessary inspection regime is acceptable. Eurocode 9 places certain conditions on the use of the damage tolerant design method and these are reinforced and supplemented in PD 6701-2.

Informative Annex B gives guidance on assessment of crack growth by fracture mechanics. This is useful and can be used in damage tolerance calculations. Other annexes that the UK considered useful and/or acceptable for use include information on fatigue testing, stress analysis, adhesive joints and the effect of stress ratio.

Other informative annexes covering low cycle fatigue, detail category tables and hot spot stresses are not recommended for use in the UK and alternative data are given in PD 6701-2.

BS EN 1999-1-4:2007 *Cold-formed structural sheeting*

Aluminium cold-formed structural sheeting has a light weight and excellent corrosion resistance and is widely used for cladding framed structures.

This Part of Eurocode 9 covers construction where cold-formed sheeting contributes to the overall strength of a structure and also in situations where it simply acts as a cladding component that only transfers load to the structure. It is therefore suitable for the full range of calculations necessary for stressed skin design and gives specific applicable rules for this application.

BS EN 1999-1-5:2007 *Shell structures*

This Part of Eurocode 9 applies to the structural design of shell or monocoque assemblies with particular reference to cylindrical, conical, torisherical and toriconical structures. The scope includes stiffened and unstiffened shells and the associated plates, section rings and stringers that form the complete structure.

The basic premise is that analysis is carried out by finite element methods to compute stresses in the shell. The code gives criteria for the treatment of geometry and boundary conditions in the finite element models.

The structure capacity is based either on yield or buckling criteria and rules are given for the use of results from a variety of types of computer analysis incorporating linear or non-linear geometric and material properties. Formulae for critical buckling stresses for simple cylindrical and conical shapes are given in an appendix. The code also allows the user to determine the critical buckling stress from linear elastic bifurcation (eigenvalue) analysis.

Reference is made to the different levels of geometric tolerances given in BS EN 1090-3 *Execution of steel structures and aluminium structures, Part 3: Technical requirements for aluminium structures* and factors are included in the formulae for buckling strength to allow for the imperfection levels as these can have a large impact on the resultant buckling strength.

Annex A. Design of an LVL garage beam conforming to BS EN 1995-1

Arnold Page, BSc, BD, MIWSc. Structural timber engineering consultant

NOTE This annex is to be read with chapter/section 5: *Eurocode 5: Design of timber structures*.

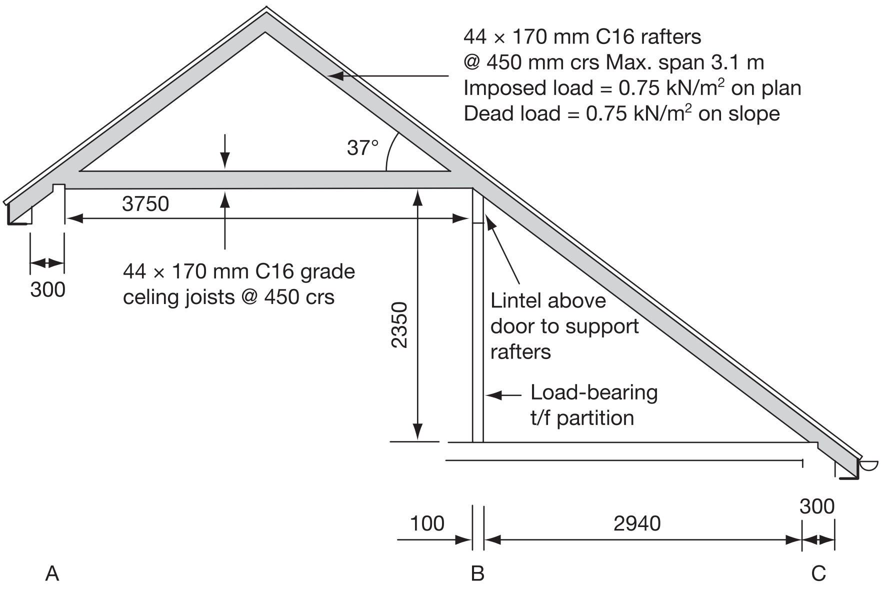


Figure A.1. Roof structure

The garage beam spans 2.58 m and is located above B, where it supports a first floor load-bearing partition and a 600 mm wide strip of the first floor.

A.1 General data

Consider an LVL beam 2580 mm long × 51 mm wide × 260 mm deep.

Material properties from BS EN 14374

$f_{m,0,edge,k}$	$= 44.0\,N/mm^2$
s (used to calculate the depth factor for bending)	$= 0.12$
$f_{v,0,edge,k}$	$= 4.1\,N/mm^2$
$E_{0,mean}$	$= 13800\,N/mm^2$
$E_{0.05}$	$= 11700\,N/mm^2$
ρ_k	$= 480\,kg/m^3$

Geometrical properties

b	$= 51\,mm$
h	$= 260\,mm$
ℓ	$= 2580\,mm$
$A = bh = 51 \times 260$	$= 13260\,mm^2$
$W = bh^2/6 = 51 \times 260^2/6$	$= 574.6 \times 10^3\,mm^3$
$I = bh^3/12 = 51 \times 260^3/12$	$= 74.70 \times 10^6\,mm^4$

Material factors for LVL (Service class 2 – unheated garage) from BS EN 1995-1-1

k_{mod} = 0.60 permanent loads
= 0.80 floor imposed – medium-term according to the UK national annex to the Eurocode
= 0.90 snow – short-term according to the UK national annex to the Eurocode

k_{def} = 0.8

k_h $= \min[(300/h)^s, 1.2]$ BS EN 1995-1-1 Clause 3.4(3)
$= \min[(300/260)^{0.12}, 1.2] = \min[1.02, 1.2] = 1.02$

γ_M = 1.2 for LVL

Load factors

For dead and imposed loads BS EN 1990 gives partial factors of 1.35 and 1.5 respectively. Partial load factors from BS EN 1990 National Annex Table A1.1 in the national annex to BS EN 1990 are as follows:

ψ_0 = 0.7 for floor imposed loads
= 0.5 for snow
ψ_1 = 0.5 for floor imposed loads
= 0.2 for snow
ψ_2 = 0.3 for floor imposed loads
= 0.0 for snow

A.2 Loads

A.2.1 Roof loads

Characteristic permanent load on roof on plan
= 0.75/cos 37° = <u>0.939 kN/m²</u>

Characteristic permanent load on roof on plan	= 0.75/cos 37°	= $\underline{0.939\,\text{kN/m}^2}$
Characteristic short-term load on roof on plan	= 0.75	= $\underline{0.75\,\text{kN/m}^2}$
Design permanent load on roof on plan	= 1.35 × 0.939	= $\underline{1.267\,\text{kN/m}^2}$
Design short-term snow load on plan	= 1.5 × 0.75	= $\underline{1.125\,\text{kN/m}^2}$

Loads on A to B (3.85 m) per metre parallel to the beam

Characteristic permanent load	= 0.939 × 3.85	= $\underline{3.62\,\text{kN/m}}$
Characteristic short-term load	= 0.75 × 3.85	= $\underline{2.89\,\text{kN/m}}$
Design permanent load	= 1.35 × 3.62	= $\underline{4.88\,\text{kN/m}}$
Design short-term load	= 1.5 × 2.89	= $\underline{4.33\,\text{kN/m}}$

Loads on B to C (3.04 m) per metre parallel to the beam

Characteristic permanent load	= 0.939 × 3.04	= $\underline{2.85\,\text{kN/m}}$
Characteristic short-term load	= 0.75 × 3.04	= $\underline{2.28\,\text{kN/m}}$
Design permanent load	= 1.35 × 2.85	= $\underline{3.85\,\text{kN/m}}$
Design short-term load	= 1.5 × 2.28	= $\underline{3.42\,\text{kN/m}}$

A.2.2 Rafters and ceiling joists

Characteristic weight of timber A to B per metre parallel to beam = $370 \times 9.81[(3850 \times 44 \times 170) + (3850 \times 44 \times 145/\cos 37°)]/[0.45 \times 10^{12}]$ = $\underline{0.480\,\text{N/mm}}$

Design permanent load from A to B per metre parallel to beam = 1.35×0.480 = $\underline{0.648\,\text{N/mm}}$

Characteristic weight of timber B to C per metre parallel to beam = $370 \times 9.81[(3040 \times 44 \times 145/\cos 37°)]/[0.45 \times 10^{12}]$ = <u>0.196 N/mm</u>

Design permanent load from B to C per metre parallel to beam = 1.35×0.196 = <u>0.265 N/mm</u>

A.2.3 First floor timber frame partition

Weight of timber frame partition (*Manual for the design of timber building structures to Eurocode 5*, IStructE/TRADA, Table 4.5) = <u>0.24 kN/m^2</u>

Hence characteristic load from timber frame partition per metre along the beam = $0.24 \times 2350/1000$ = <u>0.564 kN/m</u>

Design permanent timber frame partition load per metre on the beam = 1.35×0.564 = <u>0.762 N/mm</u>

A.2.4 First floor dead weight

22 mm chipboard @ 600 kg/m^3 = $600 \times 22 \times 9.81/10^6$ = 0.129 kN/m^2

12.5 mm firecheck plasterboard @ 850 kg/m^3 = $850 \times 12.5 \times 9.81/10^6$ = 0.104 kN/m^2

200 mm Rocksilk mineral wool @ 20 kg/m^3 = $20 \times 200 \times 9.81/10^6$ = 0.039 kN/m^2

Total dead weight = <u>0.272 kN/m^2</u>

Assuming beam replaces a floor joist spaced at 600 mm centres characteristic dead load on 600 mm = 0.6×0.272 = <u>0.163 kN/m</u>

Design permanent load from floor = 1.35×0.163 = <u>0.220 kN/m</u>

A.2.5 First floor imposed load

First floor joists at 600 mm centres support imposed floor load of 1.5 kN/m^2

Hence characteristic imposed floor load per metre on the beam = 1.5×0.6 = <u>0.90 kN/m</u>

Design medium-term imposed floor load per metre on the beam = 1.5×0.9 = <u>1.35 kN/m</u>

A.2.6. Self-weight of beam

Characteristic weight of beam = 480 × 9.81 × 13260/10^9 = 0.062 kN/m
Design permanent weight of beam = 1.35 × 0.062 = 0.084 kN/m

A.2.7 Characteristic loads on beam

$\sum G_k$, characteristic permanent load =
0.5(3.62 + 2.85 + 0.48 + 0.196) + 0.564 + 0.163 + 0.062) = 4.36 kN/m
A.2.1 A.2.1 A.2.2 A.2.2 A.2.3 A.2.4 A.2.6
$Q_{k,2}$, characteristic medium-term load *2.5* = 0.90 kN/m
$Q_{k,1}$, characteristic short-term load = 0.5(2.89 + 2.28) *A.2.1* = 2.58 kN/m

A.2.8 Design loads on beam (from A2.7)

$\sum G_d$, design permanent load = 1.35 × 4.36 = 5.89 kN/m
$Q_{d,2}$, design medium-term load = 1.5 × 0.90 = 1.35 kN/m
$Q_{d,1}$, design short-term load = 1.5 × 2.58 = 3.88 kN/m
Total permanent duration design load = 5.89 kN/m
Total medium-term design load = 5.89 + 1.35 = 7.24 kN/m
Total short-term design load = 5.89 + 1.35 + 3.88 = 11.12 kN/m

Dividing the three design values by the values of k_{mod} for the corresponding load durations (0.6, 0.8, 0.9) we obtain 9.82, 9.05 and 12.36, so it can be seen that the short-term load case is critical.

A.3 Critical short-term load case – normal design situation

A.3.1 Design values of load and strength properties

The design value of the load for the short-term load case
(BS EN 1990 (6.10)) = $\sum G_d + Q_{d,1} + \psi_{0,2} Q_{d,2}$ =
5.89 + 3.88 + 0.7 × 1.35 *A.2.8* = 10.72 kN/m

Design values of strength properties for the short-term load case

$f_{m,d} = f_{m,k} \times k_h \times k_{mod}/\gamma_M = 44.0 \times 1.02 \times 0.90/1.2$ = $\underline{33.7\,\text{N/mm}^2}$

$f_{v,d} = f_{v,k} \times k_{mod}/\gamma_M$ = $\underline{3.08\,\text{N/mm}^2}$

A.3.2 Shear force and bending moment

$F_{v,d}$, maximum design shear force = 10.72 × 2580/2 = $\underline{13830\text{ N}}$

M_d, maximum design bending moment = 10.72 × 2580²/8 = $\underline{8.92 \times 10^6\,\text{Nmm}}$

A.3.3 Shear strength

Design shear stress $\tau_d = 1.5F_{v,d}/A = 1.5 \times 13830/13260$ = $\underline{1.56\,\text{N/mm}^2}$

$f_{v,d}$ of 3.08 > 1.56, therefore shear strength is adequate

A.3.4 Bending strength

Design bending stress $\sigma_{m,d} = M_d/W_y = 8.92 \times 10^6/574.6 \times 10^3$ = $\underline{15.52\,\text{N/mm}^2}$

$f_{m,d}$ of 33.7 > 15.52, therefore bending strength is adequate

A.3.5 Bearing strength

This will be governed by the bearing area required in the blockwork wall.

A.3.6 Deflection

Serviceability load on beam (partial factors = 1.0) (BS EN 1990 (6.14b))

$p_{ser} = \Sigma G_k + Q_{k,1} + Q_{k,2} = 4.36 + 2.58 + 0.9$ = $\underline{7.84\,\text{N/mm}}$

Quasi-permanent creep load on beam (BS EN 1990 (6.16b))

$p_{creep} = k_{def}(\Sigma G_k + \psi_{2,1}Q_{k,1} + \psi_{2,2}Q_{k,2}) = 0.8(4.36 + 0 \times 2.58 + 0.3 \times 0.9)$ = $\underline{3.70\,\text{N/mm}}$

Hence final bending deflection
$= 5(p_{ser} + p_{creep})\ell^4/384E_{mean}I$
$= 5 \times (7.84 + 3.70) \times 2580^4/(384 \times 13800 \times 74.70 \times 10^6)$ = <u>6.46 mm</u>
Final deflection limit for members with attached plasterboard from the UK national annex to BS EN 1995-1-1 = $\ell/250$ = 2580/250 = <u>10.32 mm</u>
10.32 > 6.46 therefore the bending stiffness is adequate.
(A shear deflection calculation is unnecessary since shear deflection is normally < 10% bending deflection.)

The next smaller standard depth of LVL is 200 mm.
This would deflect by $6.46 \times (260/200)^3$ = <u>14.19 mm</u>
which exceeds the recommended deflection limit of 10.32 mm.

Therefore the selected 51 mm × 260 mm section is adequate.

4 Fire – accidental design situation

4.1 Charring rates

According to BS EN 1995-1-2 Clause 3.4.3.3(2), 12.5 mm thick Type A or Type F gypsum plasterboard will provide 21 min of full fire protection for any type of softwood beam. After this a beam protected by Type A plasterboard will char at a rate of 0.8 mm/min (see Table 3.1 of BS EN 1995-1-2) on all sides exposed to fire, and a beam protected by Type F (firecheck) plasterboard will char at a rate of 0.775 × 0.8 = 0.62 mm/min (BS EN 1995-1-2 Clause 3.4.3.2(1) and (2)). The beam will be designed with initial protection from fire to withstand 30 min of fire.

4.2 Charring depth

Using the reduced cross-section method in BS EN 1995-1-2 Clause 4.2.2 as specified in the UK national annex,

d_{ef} = effective charring depth on each face = $d_{char,n} + d_0$
where $d_{char,n} = \beta_{nt}$ mm

d_0 = 7 mm
β_n = notional charring rate = 0.62 mm/min as calculated above
t = time from start of charring = 9 min

Hence d_{ef} = 0.62 × 9 + 7 = 12.58 mm
So the width of 51 mm decreases by 2 × 12.58 mm to 25.84 mm.
The depth of 260 mm decreases by 12.58 mm to 247.4 mm.

4.3 Properties and factors

Geometrical properties

A_{fi} = 25.84 × 247.4 = 6393 mm^2
W_{fi} = 25.84 × 247.4^2/6 = 263.6 × 10^3 mm^3
I_{fi} = 25.84 × 247.4^3/12 = 32.61 × 10^6 mm^4

Factors for LVL (Accidental loading, fire) from BS EN 1995-1-1 and BS EN 1995-1-2

$K_{mod,fi}$ = 1.0 (BS EN 1995-1-2 Clause 4.2.2(5))
k_{def} = 0.8
$k_{h,fi}$ = min[(300/h)^s, 1.2] (BS EN 1995-1-1 Clause 3.4(3))
= min[(300/247.4)0.12, 1.2] = min[1.61,1.2] = 1.2
$\gamma_{M,fi}$ = 1.0 for LVL

Partial load factors from BS EN 1990 National Annex Table A1.1

ψ_1 = 0.2 for snow
ψ_2 = 0.3 for floor imposed loads

4.4 Design values of load and strength properties

Design load for accidental situations:
= $\Sigma G_k + \psi_{1,1}Q_{k,1} + \psi_{2,1}Q_{k,2}$ (BS EN 1990 expression (6.11b))
As before:
ΣG_k = 4.36 kN/m (dead)
$Q_{k,1}$ = 2.58 kN/m (snow)
$Q_{k,2}$ = 0.90 kN/m (floor imposed)

Hence design load for fire = 4.36 + 0.2 × 2.58 + 0.3 × 0.90 = 5.15 kN/m

20% fractile strength properties for LVL are obtained by increasing the characteristic values by a factor of 1.1 (BS EN 1995-1-2 Clause 2.3(3)).

Design values of strength properties:

$f_{m,d} = 1.1 f_{m,k} \times k_{h,fi} \times k_{mod,fi}/\psi_{M,fi} = 1.1 \times 44.0 \times 1.2 \times 1.0/1.0 = \underline{58.1\,\text{N/mm}^2}$

$f_{v,d} = 1.1 f_{v,k} \times k_{mod,fi}/\gamma_{M,fi} = 1.1 \times 4.1 \times 1.0/1.0 = \underline{4.51\,\text{N/mm}^2}$

4.5 Shear force and bending moment

$F_{v,d,fi}$, maximum design shear force = 5.15 × 2580/2 = $\underline{6644\ \text{N}}$

$M_{d,fi}$, maximum bending moment = 5.15 × 2580^2/8 = $\underline{4.29 \times 10^6\,\text{Nmm}}$

4.6 Shear strength

Design shear stress $\tau_d = 1.5 F_{v,d,fi}/A_{fi} = 1.5 \times 6644/6393 = \underline{1.56\,\text{N/mm}^2}$

$f_{v,d}$ of 4.51 > 1.56, therefore the shear strength is adequate.

4.7 Bending strength

Design bending stress $\sigma_{m,d}$ =

$M_{d,fi}/W_{fi} = 4.29 \times 106/263.6 \times 10^3 = \underline{16.27\,\text{N/mm}^2}$

$f_{m,d}$ of 58.1 > 16.27, therefore the bending strength is adequate.

4.8 Bearing strength

This will be governed by the bearing area required in the blockwork wall. BS EN 1995-1-2 states that for fire design the effects of compression perpendicular to the grain may be ignored.

4.9 Deflection

There are no recommended deflection limits for fire design according to BS EN 1995, and deformation has to be considered only where it affects the means of protection or the design criteria for separating elements (BS EN 1995-1-2 Clause 2.1.1(3)). In this case the plasterboard is required to retain its integrity for the first 21 min of a fire.

As previously calculated, in normal design the serviceability load is 7.84 N/mm and the quasi-permanent creep load is 3.70 N/mm giving a total design load of 11.54 N/mm and a final bending deflection of 6.46 mm with an I value of $74.70 \times 10^6\,mm^4$.

In fire design the reduced design load is 5.15 N/mm and the quasi-permanent creep load is 3.70 N/mm giving a total design load of 8.85 N/mm. The final I_{fi} value is $32.61 \times 10^6\,mm^4$.

By scaling the bending deflection at 21 min =
    6.46 × 8.85/11.54 = <u>4.95 mm</u>

10.32 > 4.95 so the plasterboard should not crack at this stage.

By scaling the final bending deflection in fire with a reduced
    cross-section = 4.95 × 74.70/32.61 = <u>11.34 mm</u>

This is only a little over the recommended final deflection limit of 10.32 mm previously calculated for members with attached plasterboard, so it is unlikely to cause any serviceability problem. Therefore the bending stiffness is adequate.

Therefore the selected 51 mm × 260 mm LVL beam should be protected by one layer of 12.5 mm Type F gypsum plasterboard with all joints filled before skimming.

Web contact details for further information and training

Aluminium matter
http://aluminium.matter.org.uk/content/html/eng/default.asp?catid=&pageid=1

BSI
http://shop.bsigroup.com/en/Browse-by-Subject/Eurocodes/?t=r

eurocode6.org
http://www.eurocode6.org/

eurocode7.com
http://eurocode7.com/

Eurocode expert
http://www.eurocodes.co.uk/

International Masonry Society
http://www.masonry.org.uk/

Joint Research Centre
http://eurocodes.jrc.ec.europa.eu/

The Concrete Centre
http://www.concretecentre.com/

The Concrete Society
http://www.concrete.org.uk/

The Institution of Civil Engineers
http://www.ice.org.uk/homepage/index.asp

The Institution of Structural Engineers
http://www.istructe.org/

The Society for Earthquake and Civil Engineering Dynamics
http://www.seced.org.uk/

The Steel Construction Institute
http://www.steel-sci.org/

TRADA
http://www.trada.co.uk/

The Essential Eurocodes Transition Guide

Their introduction in March 2010 requires the withdrawal of more than 50 British Standards. The change has been described as the single most important change to construction standards ever. It is expected that Eurocodes will not just dominate building design in the UK and Europe, but also have a major impact on many other parts of the world.

The Essential Eurocodes Transition Guide

In addition to coverage of the underlying structure, and industry view of the Eurocodes, individual chapters address the technical aspects of each Eurocode and will be an invaluable aid to practising civil and structural engineers, as well as regulators, academics and students and all those involved with the structure and design of public sector dwellings.

BSI order ref: BIP 2197

ISBN 978-0-580-69451-6

9 780580 694516

The British Standards Institution
is incorporated by Royal Charter