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DESIGNERS' GUIDES TO THE EUROCODES

DESIGNERS' GUIDE TO EUROCODE 6: DESIGN OF MASONRY STRUCTURES
EN 1996-1-1

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Introduction

The following introduction is intended to help the reader to gain a broad overview of the Eurocode system and the way in which it fits into the need for free trade within the European Economic Community.

Although the guide is predominantly concerned with only the main structural section – EN 1996-1-1 – the four parts of EN 1996 are as shown below.

BS EN 1996-1-1:2005	Eurocode 6. Design of masonry structures. General rules for reinforced and unreinforced masonry structures
BS EN 1996-1-2:2005	Eurocode 6. Design of masonry structures. General rules. Structural fire design
BS EN 1996-2:2006	Eurocode 6. Design of masonry structures. Design considerations, selection of materials and execution of masonry
BS EN 1996-3:2006	Eurocode 6. Design of masonry structures. Simplified calculation methods for unreinforced masonry structures

The British standards used for the design of masonry since 2005 are BS 5628 Parts 1 to 3. They deal with unreinforced masonry, reinforced and prestressed masonry and the specification and workmanship of all masonry, respectively.

Standards pertaining before the end of March 2010

BS 5628-1:2005	Code of practice for the use of masonry. Structural use of unreinforced masonry
BS 5628-2:2005	Code of practice for the use of masonry. Structural use of reinforced and prestressed masonry
BS 5628-3:2005	Code of practice for the use of masonry. Materials and components, design and workmanship

This changed at the end of March 2010. The BS 5628 suite was withdrawn, leaving BS EN 1996 as the only current code.

The way in which the different parts of BS 5628 correlate with the different parts of EN 1996 can be seen below. The correlation isn't exact – for example, BS 5628 Part 3 contains guidance on fire ratings which also correlates with Part 1-2 of EN 1996.

Pre-March 2010: BS 5628		**Post-March 2010: EN 1996**
Parts 1 and 2	now becomes	Part 1.1 and National Annex
Part 3	now becomes	Part 2 and National Annex Part 1.2 (fire) and National Annex Part 3 and National Annex

As mentioned, the British Standards Institution (BSI) stopped supporting the original British standards (BS 5628 and other structural codes) at the end of March 2010. A list of all the superseded British standards is given below.

Superseded British standards withdrawn on 31 March 2010		Superseded by
Loading for buildings		
BS 6399-1:1996	Loading for buildings. Code of practice for dead and imposed loads	EN 1991
BS 6399-2:1997	Loading for buildings. Code of practice for wind loads	EN 1991
BS 6399-3:1988	Loading for buildings. Code of practice for imposed roof loads	EN 1991
Structural use of concrete		
BS 8110-1:1997	Structural use of concrete. Code of practice for design and construction	EN 1992
BS 8110-2:1985	Structural use of concrete. Code of practice for special circumstances	EN 1992
BS 8110-3:1985	Structural use of concrete. Design charts for singly reinforced beams, doubly reinforced beams and rectangular columns	EN 1992
BS 8007:1987	Code of practice for design of concrete structures for retaining aqueous liquids	EN 1992
Structural use of steel		
BS 5950-1:2000	Structural use of steel work in building. Code of practice for design. Rolled and welded sections	EN 1993
BS 5950-2:2000	Structural use of steel work in building. Specification for materials, fabrication and erection. Rolled and welded sections	EN 1994
BS 5950-3.1:1990	Structural use of steel work in building. Design in composite construction. Code of practice for design of simple and continuous composite beams	EN 1994
BS 5950-4:1994	Structural use of steel work in building. Code of practice for design of composite slabs with profiled steel sheeting	EN 1994
BS 5950-5:1994	Structural use of steel work in building. Code of practice for design of cold formed thin gauge sections	EN 1993
BS 5950-6:1995	Structural use of steel work in building. Code of practice for design of light gauge profiled steel sheeting	EN 1993
BS 5950-7:1992	Structural use of steel work in building. Specification for materials and workmanship: cold formed sections	Obsolete
BS 5950-8:2003	Structural use of steel work in building. Code of practice for fire resistant design	EN 1993
BS 5950-9:1994	Structural use of steel work in building. Code of practice for stressed skin design	EN 1993
BS 449-2: 1969	Specification for the use of structural steel in building. Metric units	EN 1993
BS 4604-1: 1970	Specification for the use of high strength grip bolts in structural steelwork. Metric series. General grade	EN 1993
BS 4604-2: 1970	Specification for the use of high strength grip bolts in structural steelwork. Metric series. Higher grade (parallel shank)	EN 1993
Structural use of timber		
BS 5268-2:2002	Structural use of timber. Code of practice for permissible stress design, materials and workmanship	EN 1995

Superseded British standards withdrawn on 31 March 2010		Superseded by
BS 5268-3:2006	Structural use of timber. Code of practice for trussed rafter roofs	EN 1995
BS 5268-4.1:1978	Structural use of timber. Fire resistance of timber structures. Recommendations for calculating fire resistance of timber members	EN 1995
BS 5268-4.2:1990	Structural use of timber. Fire resistance of timber structures. Recommendations for calculating fire resistance of timber stud walls and joisted floor constructions	EN 1995
BS 5268-5:1989	Structural use of timber. Code of practice for the preservative treatment of structural timber	Obsolete
BS 5268-6.1:1996	Structural use of timber. Code of practice for timber frame walls. Dwellings not exceeding seven storeys	EN 1995
BS 5268-6.2:2001	Structural use of timber. Code of practice for timber frame walls. Buildings other than dwellings not exceeding seven storeys	EN 1995
BS 5268-7.1:1989	Structural use of timber. Recommendations for the calculation basis for span tables. Domestic floor joists	EN 1995
BS 5268-7.2:1989	Structural use of timber. Recommendations for the calculation basis for span tables. Joists for flat roofs	EN 1995
BS 5268-7.3:1989	Structural use of timber. Recommendations for the calculation basis for span tables. Ceiling joists	EN 1995
BS 5268-7.4:1989	Structural use of timber. Ceiling binders	EN 1995
BS 5268-7.5:1990	Structural use of timber. Recommendations for the calculation basis for span tables. Domestic rafters	EN 1995
BS 5268-7.6:1990	Structural use of timber. Recommendations for the calculation basis for span tables. Purlins supporting rafters	EN 1995
BS 5268-7.7:1990	Structural use of timber. Recommendations for the calculation basis for span tables. Purlins supporting sheeting or decking	EN 1995
Structural use of masonry		
BS 5628-1:2005	Code of practice for the use of masonry. Structural use of unreinforced masonry	EN 1996
BS 5628-2:2005	Code of practice for use of masonry. Structural use of reinforced and prestressed masonry	EN 1996
BS 5628-3:2005	Code of practice for use of masonry. Materials and components, design and workmanship	EN 1996
Geotechnics		
BS 8002:1994	Code of practice for earth retaining structures	EN 1997-1
BS 8004:1986	Code of practice for foundations	EN 1997-1
Structural use of aluminium		
BS 8118-1:1991	Structural use of aluminium. Code of practice for design	EN 1999
BS 8118-2:1991	Structural use of aluminium. Specification for materials, workmanship and protection	EN 1999
Bridges		
BS 5400-1:1988	Steel, concrete and composite bridges. General statement	EN 1990, EN 1991
BS 5400-2:2006	Steel, concrete and composite bridges. Specification for loads	EN 1990, EN 1991
BS 5400-3:2000	Steel, concrete and composite bridges. Code of practice for design of steel bridges	EN 1993
BS 5400-4:1990	Steel, concrete and composite bridges. Code of practice for design of concrete bridges	EN 1992

Superseded British standards withdrawn on 31 March 2010		Superseded by
BS 5400-5:2005	Steel, concrete and composite bridges. Code of practice for design of composite bridges	EN 1994
BS 5400-6:1999	Steel, concrete and composite bridges. Specification for materials and workmanship, steel	EN 1090-2
BS 5400-7:1978	Steel, concrete and composite bridges. Specification for materials and workmanship, concrete, reinforcement and prestressing tendons	EN 1992
BS 5400-8:1978	Steel, concrete and composite bridges. Recommendations for materials and workmanship, concrete, reinforcement and prestressing tendons	EN 1992
BS 5400-9.1, BS 5400-9.2	Bearings	Not affected

The EN 1996 suite of codes uses the 'new' set of European EN (European Norm) supporting standards (for units, mortars, ancillary components, etc.) and not the previous British standards. The 2005 edition of BS 5628 Parts 1 to 3 also used the new set of supporting standards. A complete raft of European-wide supporting standards has been prepared, and the structural Eurocode for masonry can refer to them across every EEC member country using the Eurocodes; some of the important standards are shown below.

Some of the main supporting standards on which EN 1996 relies

EN 771	Units
EN 772	Tests for units
EN 845	Ancillary components
EN 846	Tests for ancillary components
EN 998	Masonry mortars
EN 1015	Tests for mortars
EN 1052	Tests for masonry

Each of the above European Norms has several parts. In the case of EN 1015 Part 21, 'Methods of test for mortar for masonry. Determination of the compatibility of one-coat rendering mortars with substrates', clearly defines the contents. EN 772, 'Methods of test for masonry units', contains 22 parts, although only 17 have been published. (It was originally believed that 22 parts – or test methods – would be necessary, and these were prepared. EN 771, however, when finalised, called on only 17 tests. The other five parts, not referred to in EN 771, were not published.) EN 771, 'Specification for masonry units', has six parts covering the major different units commonly used in masonry construction. EN 771-1, 'Specification for masonry units – Part 1: Clay masonry units', was introduced in 2003: its title clearly defines its scope. Before that date, 'Clay masonry units' – essentially, clay bricks – was specified as BS 3921. The complete list of masonry unit standards is shown below – together with some of the 'old' BS numbers that used to pertain.

Masonry unit standards

EN 771-1: Specification for masonry units – Part 1: Clay masonry units	BS 3921
EN 771-2: Specification for masonry units – Part 2: Calcium silicate masonry units	BS 187
EN 771-3: Specification for masonry units – Part 3: Aggregate concrete masonry units (dense and light-weight aggregates)	BS 6073
EN 771-4: Specification for masonry units – Part 4: Autoclaved aerated concrete masonry units	BS 6073
EN 771-5: Specification for masonry units – Part 5: Manufactured stone masonry units	BS 6457
EN 771-6: Specification for masonry units – Part 6: Natural stone masonry units	None

The suite of Eurocodes

EN 1996 is one title in a suite of Eurocodes.

The structural Eurocodes
Eurocode 0: Basis of design
Eurocode 1: Actions on structures
Eurocode 2: Design of concrete structures
Eurocode 3: Design of steel structures
Eurocode 4: Design of composite steel and concrete structures
Eurocode 5: Design of timber structures
Eurocode 6: Design of masonry structures
Eurocode 7: Geotechnical design
Eurocode 8: Design provisions for earthquake resistance of structures
Eurocode 9: Design of aluminium alloy structures

Eurocode 0 covers the basis of design while Eurocode 1 is on actions on structures (i.e. loading). The titles of the other Eurocodes are fairly self-explanatory. At the time of writing, there is discussion about the need to prepare a Eurocode on the design of structural glass: presumably, if the mandate is given, this will become Eurocode 10.

What is the concept?

Much has been written on the historical development of the common market and the European Union, such as the names and dates of important treaties that formed major milestones in the growth and development of today's modern Europe. Because this is widely covered elsewhere, it does not appear relevant to continue on this track: suffice it to say that the concept behind the Eurocodes was not a technical one but a political idea born of political thinking about how the countries of Europe and the societies they contained should relate to themselves and to each other in the second half of the twentieth century and beyond. Modern society being complex, this covered many aspects. One aspect, along with others such as *food* and *transport,* was *construction*. While these terms were used in the 'broadest' of senses to describe *sectors*, one aspect of the *construction sector* is the design and construction of new buildings and other (infrastructure) works. The concept was simply that one design code should be available for each structural material – such as masonry – through all the member countries of the European Committee for Standardization (Comité Européen de Normalisation – CEN), from Iceland to Greece and from Finland to Spain. The aim as stated by the European Commission was 'to establish a set of common technical rules for the design of buildings and civil engineering works which will ultimately replace the differing rules in the various Member States'. When this is viewed in the knowledge that masonry units and mortars are covered in all the countries by the same European Norms (EN 771, etc.), it can be seen that a level playing field for trade is achieved when viewing the design profession as a whole as well as the masonry material producers.

A UK consultant designing a building in their own town and one in Madrid and one in Latvia will now require only one structural masonry code BS EN 1996-1-1. The rules in this code EN 1996-1-1 apply to Spain and Latvia as well as the UK.

How do the Eurocodes work?

EN 1996 was prepared in a pan-European CEN committee with the draft being developed in English. CEN – the European Committee for Standardization – has as its members the national standards bodies (NSBs) of European nations. The member countries of CEN are shown below.

National members of CEN
The NSBs of the 31 member countries of CEN are shown below. There is one member per country (BSI for the UK).

Austria	Belgium	Bulgaria	Croatia	Cyprus	Czech Republic
Denmark	Estonia	Finland	France	Germany	Greece
Hungary	Iceland	Ireland	Italy	Latvia	Lithuania
Luxembourg	Malta	Norway	Poland	Portugal	Romania
Slovakia	Slovenia	Spain	Sweden	Switzerland	The Netherlands
UK					

At appropriate milestones, such as the completion of the final draft of the code in English, translations into French and German were prepared/finalised and all three language versions were distributed to all NSBs. The French and German translations were prepared with great care to ensure that they contained exactly the same information and design rules as the original draft in English. When signed off as 'complete and agreed' by the relevant committee, EN 1996 was released by CEN to all the NSBs in CEN membership. When so released, NSBs published EN 1996. In the UK, BSI published it in English as BS EN 1996. In Germany (through DIN) it was published in German as DIN EN 1996, and in France it was published (through AFNOR) in French as NF EN 1996. Some countries, such as Finland or Sweden for example, need to translate all the Eurocodes (not just EN 1996) into their own language. Equally, some countries have published the documents in English as well as in their own mother tongues. Some countries have decided that they will translate into their own language only those parts of EN 1996 that they believe will be used in their country. The writer is aware of one country that will not be publishing all of the parts of EN 1996 in its own language: there is no plan for one part to be translated because it is believed it will simply not be used.

What is a National Annex?

To reach agreement within a European committee is a major achievement: indeed, the vast majority of guidance contained in EN 1996 *has* been agreed by all CEN member countries – which in itself has required much co-operation. Some issues are less contentious than others. All member countries, for example, agreed that masonry should be laid in a bonded pattern for structural integrity. They also agreed on the minimum 'overlap' by which units require to be bonded to achieve this structural integrity.

Sometimes, however, the issues are such that it is not possible to reach agreement. When this occurs, it simply has to be recognised that agreement is not feasible and some other path/route has to be made available so that it can then come into play. For example, there can be country-specific information that has to do with geography or climate. This can often be hidden away in the historical/traditional ways in which construction matters have developed in a particular country. It would be totally wrong for the members to agree that the way things have been done in, say, the UK or in Germany for several decades should now be disallowed because of a new design code.

There is also the issue of safety. Most governments would be reluctant to abrogate their responsibility for the general level of safety of their stock of buildings and civil engineering works and vest that responsibility in some other European organisation. The path chosen to deal with such areas where common agreement was not possible was to accept that *country-specific information* is required in *all* the Eurocodes. When this is the case, the country-specific information is referred to as *nationally determined parameters* (NDPs). Such country-specific information (the collection of NDPs) is contained in a National Annex (NA). All CEN member countries require a National Annex for each of the four parts of EN 1996. Each National Annex applies only to buildings designed to their standard. The UK National Annex to BS EN 1991-1-1 applies to designs that require to comply with UK building regulations or designs in any other country that uses British standards. Similarly, the National Annex to NF EN 1991-1-1 will be used in France and any other country using French standards. With regard to the level of safety in a particular country, for example, the values of safety factors are one of the NDPs in the National Annex.

National Annexes are published separately: they are not bound into the code. A UK consultant designing a building in their own town or city and one in Madrid and in Latvia will now require the National Annex for EN 1996-1-1 for the UK, the National Annex for Spain and the National Annex for Latvia; on the other hand, only the one structural masonry code BS EN 1996-1-1 will be required. (Of course, the National Annexes for Spain and for Latvia may require to be translated.)

Informative or normative?

There are two types of annex used in the structural Eurocodes. One is *informative*, and is provided for information: it is not a mandatory part of the code. The other is *normative*, and is a mandatory part of the code.

It is possible for a member country to permit or not to permit any *informative* annexes to be used. The National Annex for a particular country will state which of the informative annexes may be used and which may not be used.

Principles and application rules?

Principles outline, in a clause, requirements for which there is no alternative. They are general statements, usually including the verb '*shall*', which lay down a definition or an essential condition that requires to be met. In addition to their use of 'shall', principle clauses can be easily recognised since they contain the upper-case letter 'P' immediately after the clause number and before the clause.

Application rules can be considered to be routes that fulfil the principles and satisfy the requirements. They are generally well known, tried and tested design approaches that the designer will doubtless recognise and of which they will be aware. The verb 'should' is usually used in application rule clauses, as is the verb 'may'.

If a design to EN 1996 is conducted using the relevant application rules that fulfil the relevant principles required, the design can be said to be to Eurocode EN 1996.

There may be occasions, however, when there is no suitable application rule in the code for a particular design problem posed. In such situations, it is permissible to use alternative application rules (e.g. from textbooks or published papers). If this is done, it can be argued that the design is still to BS EN 1996 – provided that the design rules used fully comply with the requirements of the relevant principle clauses.

The Published Document (PD) 6697

While the four parts of Eurocode 6 cover most of the important aspects of masonry design and construction, there are certain difficulties involved in attempting to provide full design guidance over an area extending from Finland to Spain and Iceland to Greece. We have already covered one such difficulty for which the concept of the National Annex was introduced to provide a workable way forward.

While the laws of science and engineering do not of course vary in different countries, other aspects concerning the way in which masonry is traditionally used and specified may differ. For example, the ways in which masonry materials need to be specified and constructed to enjoy satisfactory durability performance may well vary in different countries. While the general concepts of durability are fully covered in EN 1996, the detailed clauses on how to specify materials for durability are not fully covered. Similarly, in the UK there is an established practice to occasionally use bed joint reinforcement in large panels that would otherwise not work. This, too, is not as fully covered in the EN 1996 as might be desired for UK practice.

As part of the CEN procedures and quite separate from the National Annex, a country is able to publish *non-contradictory and complementary information* (NCCI) in handbooks and also in the NSB's publications. PD 6697 (from BSI) will contain any *residual* material from BS 5628 Parts 1 to 3 that complements the contents of EN 1996 and does not contradict it. (Clearly, it goes without saying that there is no benefit to be gained from duplicating information in the PD that is already in EN 1996: so that is never done.)

PD 6697 has been published. It contains, among other things, a table covering the durability of masonry materials for use in the UK as well as the four methods, which are currently in

BS 5628 Part 2, for designing laterally loaded wall panels. It also covers *accidental damage* provision.

In this way, PD 6697 can be seen to be an additional part of the Eurocode when designing in masonry in the UK or, indeed, in other areas of the world where it is the custom to base building design on British standards.

The way in which the different parts of BS 5628 correlate with the different parts of EN 1996 needs to recognise this additional document. The correlation given earlier needs amending, and the final correlation is shown below. As mentioned earlier, it is not exact.

Pre-March 2010: BS 5628		**Post-March 2010: EN 1996**
Parts 1 and 2	is covered by	Part 1.1 and National Annex and PD
Part 3	is covered by	Part 2 and National Annex and PD Part 1.2 (fire) and National Annex and PD Part 3 and National Annex (this has no real correlation to the BS 5628 suite of codes)

National Annex to EN 1996-1-1

On page 10 of EN 1991-1-1, the clauses are listed in which national choice is allowed: there are 16. CEN rules mean that the UK National Annex must address this list of clauses and give values for the NDPs referred to in each of the 16 listed clauses. This applies to all member countries of CEN. The National Annex of every country must deal with the same 16 issues although, of course, each country may have different values for some or all of the NDPs to those given in the National Annex for UK.

No country can introduce any new material in their National Annex additional to these 16 points and no country can deal with, say, 14 of them and not deal with the remainder. The rules are quite clear. All the NDPs for those clauses listed must be addressed – no more and no less. Of course, it may be that one clause generates several NDPs. For example, the clause regarding ultimate limit states is addressed in the UK National Annex by the use of a table of γ_M partial factors that has several possible values, depending on:

- the type of stress being considered
- the class of execution
- the category of unit being used.

Clearly, every reader is aware that the latest version of the code should always be used. Since a National Annex can also change or be amended, it is also important to ensure that the National Annex being used contains the latest amendments – if any.

Finally, at the end of the National Annex two other issues are resolved:

- the decision is given regarding which of the informative annexes may be used and which may not be used
- recognised NCCI is listed.

By inspection, the reader will see that all the annexes in EN 1996-1-1 are informative and, from clause NA 3 of the National Annex for Part 1-1, it can be seen that all the informative annexes may be used in the UK.

The reader will also see that PD 6697 and this guide (and others) are referenced within the list of NCCI.

Corrigenda

It should be noted that a corrigendum has been prepared for EN 1996 and distributed to the NSBs in CEN membership. This generally covers typographical errors and typographical omissions. Although the corrigendum is important, the reader should be aware that it does not make fundamental amendments that change the way that the code is technically used.

Preface

The first draft of this guide was written several years ago. It lay unworked for over two years, awaiting the preparation of the National Annexes which are an important part of all four parts of BS EN 1996, 'Design of masonry structures'. Without the National Annex for Part 1-1, 'General rules for reinforced and unreinforced masonry structures', the usefulness of this guide would be greatly reduced.

It is the intention of this document to complement BS EN 1996-1-1 by explaining the areas where change has overtaken the traditional approach used in the former British standard. Thus, the guide is intended to be non-contradictory complementary information (NCCI).

The aim of the guide is to explain dispassionately how the code writers intended the clauses to be applied.

EN 1996-1-1 provides the structural rules for designing a composite material comprising masonry units and mortar. Certain parts of this code call necessarily on EN standards for clay or concrete units, sand or cement. Where it is deemed necessary guidance is given on material aspects, particularly when the approach adopted in the EN standards is different from the approach that designers have been used to in the previous British standard version. An example of this is the method for determining f_k, the characteristic compressive strength of masonry. This requires an understanding of the EN standards for masonry units. While it is not strictly part of EN 1996-1-1 it is, nonetheless, essential to adequately cover this topic.

I have endeavoured to restrict the content of this guide to masonry construction in common use. This has been done subjectively from the writer's experience. Greater emphasis is therefore given to unreinforced masonry than to reinforced masonry, to reflect the relative usage made in practice.

A short section has been added on the subject of accidental damage which, strictly speaking, is outside the remit of EN 1996-1-1 but has been included for completeness and to assist in meeting compliance with the UK regulations on disproportionate collapse.

Layout of this guide

The section numbering in this guide matches that in EN 1996-1-1: this may, on occasion, result in anomalies in the numbering when a clause in the Eurocode does not warrant a headed section in this guide. Citations in the margin of this guide refer to numbered clauses, equations, etc. in EN 1996-1-1 that relate to the adjacent text ('NA' refers to material (e.g. Table NA.1) in the UK National Annex of EN 1996-1-1).

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I am deeply indebted to all fellow members of the BSI Mirror Committee B/525/6. Their experience, contribution to debates and helpful support has greatly assisted me with the writing of this guide.

I am also indebted to Dr G. J. Edgell, Ceram; Mr G. T. Harding (formerly of ODPM, London – now CLG) and J. N. Tutt (Jenkins and Potter), who all readily agreed to review the guide and pick up the inevitable errors and corrections that somehow creep into a text that involves calculations and words.

I also thank Professors Johnson and Anderson for their support and permission to allow me to adopt a similar format for the common Chapters 1 and 2 to that used in their *Designers' Guide to Eurocode EN 1994-1-1*.

Finally, it is always a risk highlighting only a few names in case those not given special mention are offended, but I am particularly grateful to B. A. Haseltine, Chairman of B525/6, and Professor Emeritus J. J. Roberts, Chairman of B525/6/102.

Lightwater, Surrey
June 2011

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Chapter 1
General

This chapter is concerned with the general aspects of EN 1996-1-1, 'Eurocode 6: Design of masonry structures, Part 1.1: Common rules for reinforced and unreinforced masonry structures'. The material described in this chapter is covered in *Section 1* in the following clauses:

- Scope *Clause 1.1*
- Normative references *Clause 1.2*
- Assumptions *Clause 1.3*
- Distinction between principles and application rules *Clause 1.4*
- Definitions *Clause 1.5*
- Symbols. *Clause 1.6*

1.1. Scope *Clause 1.1*

1.1.1 Scope of Eurocode 6 *Clause 1.1.1*

The scope of EN 1996 (all four parts) is outlined in clause 1.1.1. It is reasonably straightforward.

The other parts of EN 1996 that supplement Part 1-1 are: *Clause 1.1.3*

- Part 1-2: 'General rules – Structural fire design'
- Part 2: 'Design, selection of materials and execution of masonry'
- Part 3: 'Simplified calculation methods for unreinforced masonry structures'.

EN 1996 does not cover requirements for:

- thermal and sound insulation
- seismic design (covered in Eurocode 8, *Design of Structures in Seismic Regions*)
- evaluating the size of actions (covered in Eurocode 1, *Actions on Structures*, and its National Annex).

The basis for verification of safety and serviceability is the partial factor method. EN 1990 recommends values for load factors and gives various possibilities for combinations of actions. The values and choice of combinations are set by the National Annex of EN 1990 for the country in which the structure is to be constructed.

It is noted, therefore, that while γ_M values (required to calculate the load resistance of members) are NDPs and are therefore under the control of EN 1996 – or, to be pedantically correct, the National Annex for EN 1996-1-1 – the values for γ_f are not: they are governed by the National Annex for EN 1990 and, as such, the values are material independent.

The Eurocodes are concerned with design and not execution, but minimum standards of workmanship are required to ensure that the design assumptions are valid. (Execution is covered more fully in EN 1996-2.)

1.1.2 Scope of Part 1-1 of Eurocode 6 *Clause 1.1.2*

This document gives the principles of the design of prestressed and confined masonry but *not* the application rules. For reinforced and unreinforced masonry, the principles *and* the application rules are given. *Clause 1.1.2(1)P*

It is noted that Part 1-1 is not valid for masonry with a plan area of less than 0.04 m^2. In practical terms, this should not prove an obstacle.

Clause 1.1.2(2)

The code recognises that where new boundaries are tested or breached by innovation, new materials, new thinking or new experimental evidence, the information in the code may not, of itself, be sufficient and may require to be supplemented. It hints, however, that the principles and application rules may be applicable.

Clause 1.1.2(3)

Any limits deemed necessary to any of the detailed rules are given in the text.

Clause 1.1.2(4)

Clause 1.1.2(4) lists the titles of the sections of Part 1.1. Those for Sections 1–7 are the same as in the other material-dependent Eurocodes. The contents of Sections 1 and 2 similarly follow an agreed model.

Clause 1.1.2(5)

Clause 1.1.2(5) lists the topics that are specifically excluded. These are specific enough not to require further comment here.

Clause 1.1.3

1.1.3 Further parts of Eurocode 6

In clause 1.1.3, the reader is reminded of the three other parts of EN 1996 that have been mentioned above.

Clause 1.2

1.2. Normative references

References are given only to other European standards, all of which are intended to be used as a package. Formally, the standards of the International Organization for Standardization (ISO) apply only if given an EN ISO designation. National standards for design and for products do not apply if they conflict with a relevant EN standard.

It has always been intended that, following a period of overlap, all competing national standards would be withdrawn by around 2010. Although target dates can slip, this has happened and BS EN 1996 is the only current code. As Eurocodes may not cross-refer to national standards, replacement of national product standards by EN or ISO standards is essential: this has happened in the UK. In addition, until EN 1996 is issued by the various national standard bodies, national codes may be or are being suitably amended to incorporate reference to the new EN masonry material standards. The majority of designers will have some working knowledge of the new masonry material standards.

During the period of changeover to Eurocodes and EN standards it is possible that, in some countries, an EN standard referred to, or its National Annex, may not be introduced/complete. As previously mentioned, in the UK all supporting EN standards together with the National Annexes and the Published Document are now in place. Designers who do seek guidance from national standards should take account of any differences between the design philosophies and safety factors in the two sets of documents. In many ways, this is less likely to be a problem since the switch to using Eurocodes by designers is somewhat less and somewhat later than had perhaps been originally envisaged.

Clause 1.2.1

1.2.1 General reference standards

Clause 1.2.1 sets down the rules regarding which version of a referenced publication should be used. For undated references, the latest edition should be used; for dated references, the version cited should be used *unless* it has been specifically modified by amendment or revision. The general rule applies that the latest amended version of a code or National Annex should always be used.

Clause 1.2.2

1.2.2 Reference standards

The standards to which Eurocode 6 refers are listed in clause 1.2.2.

Clause 1.3

1.3. Assumptions

The general assumptions are those of EN 1990 – particularly clause 1.3.

Clause 1.4
Clause 1.4.1(P)

1.4. Distinction between principles and application rules

Clauses in the Eurocodes are set out as either principles or application rules. As defined by EN 1990:

- 'Principles comprise general statements for which there is no alternative and requirements and analytical models for which no alternative is permitted unless specifically stated.'
- 'Principles are distinguished by the letter "P" following the paragraph number.'
- 'Application rules are generally recognised rules which comply with the principles and satisfy their requirements.'

There are many fewer principle clauses than other clauses. It is generally recognised that a requirement or analytical model for which 'no alternative is permitted unless specifically stated' can rarely include a numerical value, because most values are influenced by research and/or experience and may change over the years. Furthermore, a clause cannot be a principle if it requires the use of another clause that is an application rule: effectively, the latter clause would also require to become a principle.

It follows that, ideally, the principles in all the codes should form a consistent set, referring only to each other, and intelligible if all the application rules were deleted.

1.5. Terms and definitions

Clause 1.5

1.5.1 General

Clause 1.5.1

In accordance with the model for Section 1 of EN 1996-1-1, reference is made to the definitions given in clause 1.5 of EN 1990: these definitions are then enabled by the next clause. *Clause 1.5.1(1)* *Clause 1.5.1(2)*

Next, the definitions of terms used within the code are given. These are grouped under the following headings:

- Masonry *Clause 1.5.2*
- Strength of masonry *Clause 1.5.3*
- Masonry units *Clause 1.5.4*
- Mortar *Clause 1.5.5*
- Concrete infill *Clause 1.5.6*
- Reinforcement *Clause 1.5.7*
- Ancillary components *Clause 1.5.8*
- Mortar joints *Clause 1.5.9*
- Wall types *Clause 1.5.10*
- Miscellaneous. *Clause 1.5.11*

Depending on the reader's knowledge of masonry, these are all worth at least some attention. Of particular value might be the mortar terms with which many readers are probably not very familiar. A less than familiar term is 'double-leaf wall'. This is the new term to describe what has in the UK been generally known as a 'collar jointed wall', with which many readers will be more familiar. In a similar vein, there is now no (definition for an) engineering brick. The old definition for such bricks was:

- Engineering brick A: a brick with a water absorption less that 4.5% *and* a mean compressive strength of 70 N/mm^2 or greater.
- Engineering brick B: a brick with a water absorption less that 7% *and* a mean compressive strength of 50 N/mm^2 or greater.

Designers are still able to achieve the above specification, but now need to use the water absorption and strength limits and not the old term 'engineering brick', which is no longer recognised as a defined term.

1.6. Symbols

Clause 1.6(1)

There is little to say about the symbols: they are the given starting point and, as such, will merely require those involved in calculations to become familiar with them. *Clause 1.6(2)*

Occasionally, we come across a symbol with which we should be familiar and that is actually different. For example, BS 5628 used f_{kx} for the flexural strength of masonry: this term is now f_{xk}. We will simply have to become familiar with minor changes such as this.

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Chapter 2
Basis of design

The material described in this chapter is covered in *Section 2* of EN 1996-1-1, in the following clauses:

- Basic requirements *Clause 2.1*
- Principles of limit states design *Clause 2.2*
- Basic variables *Clause 2.3*
- Verification by the partial factor method *Clause 2.4*
- Design assisted by testing *Clause 2.5*

2.1. Basic requirements

Clause 2.1
Clause 2.1.1
Clause 2.1.2
Clause 2.1.3

Design is to be in accordance with the general requirements of EN 1990. The purpose of Section 2 is to give supplementary provisions for masonry structures.

Clause 2.1.1(1)P
Clause 2.1.1(3)

Clause 2.1.1(3) reminds the user again that design is based on actions and combinations of actions in accordance with EN 1991 and EN 1990, respectively. The use of partial factors for actions and resistances (the 'partial factor method') is expected but is not a requirement of the Eurocodes. The method is presented in Section 6 of EN 1990 as one way of satisfying the basic requirements set out in Section 2 of that standard. This is why use of the partial factor method is given 'deemed to satisfy' status in clause 2.1.1(3). To establish that a design was in accordance with the Eurocodes, the user of any other method would normally have to demonstrate, to the satisfaction of the regulatory authority and/or the client, that the method satisfied the basic requirements of EN 1990.

Clause 2.1.1(3)

2.2. Principles of limit states design

Clause 2.2
Clause 2.2.1
Clause 2.2.2
Clause 2.2.3

Clause 2.2 provides a reminder that the influence of the sequence of construction on action effects must be considered, as must the ultimate and serviceability limit state for not only the masonry but also all the many other materials/components normally found in masonry construction.

2.3. Basic variables

Clause 2.3
Clause 2.3.1
Clause 2.3.2
Clause 2.3.3

Clause 2.3 merely directs the user to EN 1990, EN 1991 and EN 1992-1-1 for determining the value of actions and partial factors for both actions and creep and shrinkage of concrete elements in masonry structures.

2.4. Verification by the partial factor method

Clause 2.4
Clause 2.4.1
Clause 2.4.2

Clauses 2.4.1 and 2.4.2 illustrate the treatment of partial factors. Recommended values are given in Notes, in the hope of eventual convergence between the values for each partial factor that will be specified in the National Annexes. This process was adopted because the regulatory bodies in the member states of CEN, rather than CEN itself, are responsible for setting safety levels. The Notes are informative, not normative (i.e. not part of the preceding provision), so that there are no numerical values in the principles of clause 2.4.2, as explained earlier.

The usual convention is used: *characteristic* values will be unfactored whereas *design* values will have been factored.

Clause 2.4.3

Clause 2.4.3 makes the principle that the relevant value for the partial factor, γ_m, should be used. Again, no values are given in the clause but instead in an informative note, where recommended values are given. Each NSB, however, is given the freedom to choose values for itself and to state them in the National Annex for that country.

Clause 2.4.4

The same concept applies to serviceability limit states when detailed serviceability calculations are required. The relevant value of γ_m will be found in a country's National Annex.

Commentary

Table NA.1

Table NA.1 (Table 2.1) in the National Annex gives values of γ_M for the ultimate limit state. There are four values given for unreinforced masonry in direct or flexural compression. These are for two classes of execution control and two categories of units. The reader will immediately note that these values are different from the *default values* given in the table (within the note on page 29) of EN 1996-1-1 – this table should *not* be used: γ_M should be obtained from Table NA.1.

The values given in Table NA.1 follow a similar approach adopted in BS 5628, which was based on the concept that if there was more certainty about

- the strength of the units
- the strength of the mortar
- the way in which the mortar and units are put together on site,

there should be a corresponding bonus in terms of the amount of load that the masonry can then be permitted to carry.

This approach has been carried across into Eurocode 6. The values were decided by the BSI mirror masonry committee, which took into account a series of comparative studies that were conducted for just this purpose. They compared the outcome of calculations done using BS 5628 to the outcome of identical calculations done using Eurocode 6. In this way, the value of γ_M was found that, when doing calculations to Eurocode 6, gives the same level of safety as was present when identical calculations were done to BS 5628.

These studies were conducted only on compressive loaded walls: no studies were conducted on lateral loaded panels or on reinforced masonry.

Table NA.1

Table NA.1 then gives two values for γ_M for reinforced masonry – both using execution control class 1 since class 2 is not deemed appropriate for reinforced masonry construction.

For flexural tension, two values of γ_M are given: it is reasonable not to expect any particular increase in the design value of flexural tension/adhesion simply because the strength of the units is more certainly known.

Other aspects are then dealt with, such as anchorage of steel reinforcement and wall ties and straps.

The reader should be aware that the code committee is agreed that a form of amendment be issued that addresses the following problem. In BS 5628 the load factor γ_F is reduced by $\sim$15% (γ_F is reduced to 1.2 from 1.4) under the particular condition where the removal of a laterally loaded wall panel in no way affects the stability of the remaining structure. A lower design load can therefore be used for the flexural design of masonry wall panels to steel or concrete-framed buildings, for example.

In Eurocode 6, it is not possible to effect any change to the loading side of the design through the National Annex to EN 1996-1-1. It is possible, however, to change the value of γ_M and, in time, that is what is proposed. An additional two values will be offered which will be $\sim$15% lower. These two additional values of γ_M for flexural tension will only be applicable to the case when the removal of the wall being designed would in no way affect the stability of the remaining structure. This should give Eurocode 6 a similar benefit for this type of panel to that previously enjoyed when designing using BS 5628. The reader should be aware, however, that it is never prudent to 'jump the gun'. This is mentioned here, since an amendment to the National Annex to BS EN 1996-1-1 may have been issued by the time this guide is published or read.

Table NA.1

Table 2.1. Values of γ_M for the ultimate limit state

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[d]	3.5[b]
Ancillary components – straps	1.5[e]	1.5[e]
Lintels in accordance with EN 845-2	See the National Annex to BS EN 845-2	See the National Annex to BS EN 845-2

[a] Class 1 of execution control should be assumed whenever the work is carried out following the recommendations for workmanship in BS EN 1996-2, including appropriate supervision and inspection, and in addition:

- the specification, supervision and control ensure that the construction is compatible with the use of the appropriate partial factors given in BS EN 1996-1-1
- the mortar conforms to BS EN 1998-2, if it is factory made mortar, or if it is site mixed mortar, preliminary compression strength tests carried out on the mortar to be used, in accordance with BS EN 1015-2 and BS EN 1015-11, indicate conformity to the strength requirements given in BS EN 1996-1-1 and regular testing of the mortar used on site, in accordance with BS EN 1015-2 and BS EN 1015-11, shows that the strength requirements of BS EN 1996-1-1 are being maintained.

Class 2 of execution control should be assumed whenever the work is carried out following the recommendations for workmanship in BS EN 1996-2, including appropriate supervision.

[b] When considering the effects of misuse or accident these values may be halved.

[c] Class 2 of execution control is not considered appropriate for reinforced masonry and should not be used. However, masonry wall panels reinforced with bed joint reinforcement used:

- to enhance the lateral strength of the masonry panel
- to limit or control shrinkage or expansion of the masonry

can be considered to be unreinforced masonry for the purpose of class of execution control and the unreinforced masonry direct or flexural compression γ_M values are appropriate for use.

[d] When considering the effects of misuse or accident these values should be taken as 1.0.

[e] For horizontal restraint straps, unless otherwise specified, the declared ultimate load capacity depends on there being a design compressive stress in the masonry of at least 0.4 N/mm^2. When a lower stress due to design loads may be acting, for example when autoclaved aerated concrete or lightweight aggregate concrete masonry is used, the manufacturer's advice should be sought and a partial safety factor of 3 should be used.

Clause 2.5

2.5. Design assisted by testing

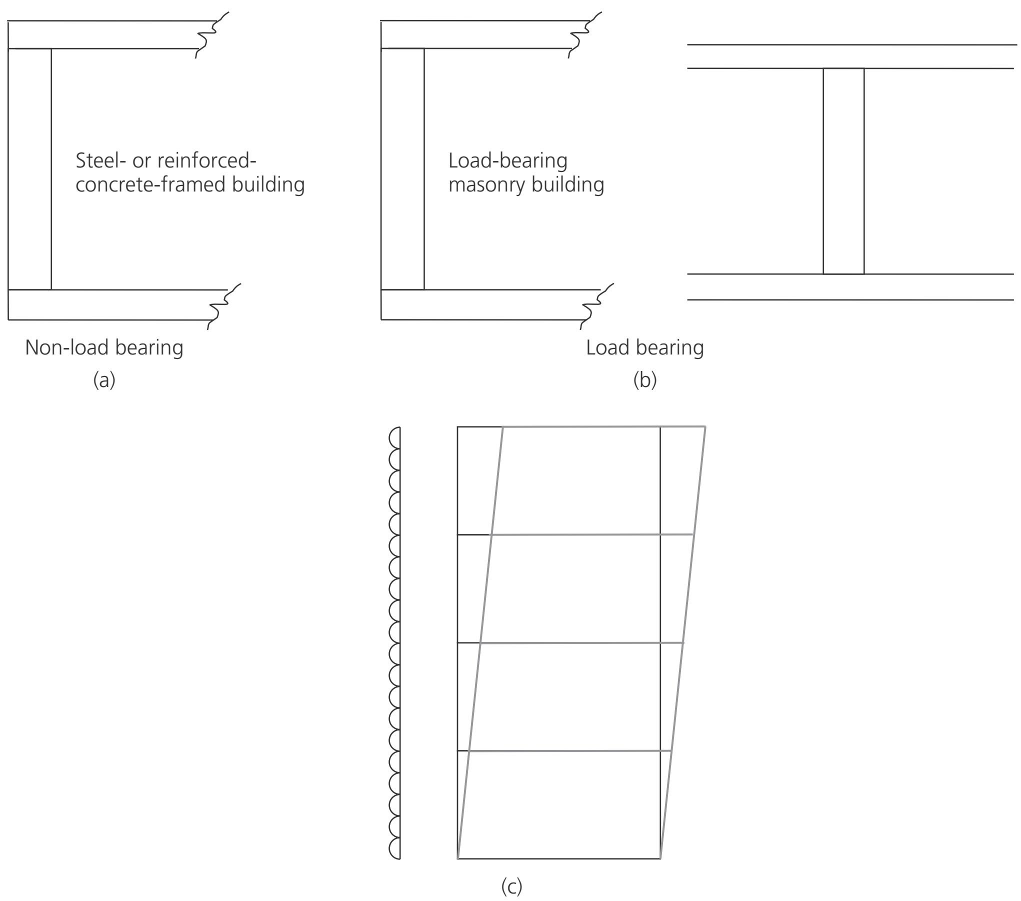
The user's attention is drawn to the fact that when the design of a particular member or sub-assembly of members appears to fall outside the range of design covered by EN 1996-1-1, structural properties may be determined by testing. The reader is referred back to EN 1990.

Commentary on Chapter 2
The basis of design – loading

While the essence of a guide on Eurocode 6 has to be masonry, the changes introduced to the loadings side of the design equations cannot be overlooked. While the reader is referred to relevant publications on Eurocode 0 and Eurocode 1, this guide would be incomplete without reference to how these changes to the loading approach can affect masonry design.

Considering the most usual forms of designing met in practice, the common forms of loading and calculation checks required are for wind-loaded panels, external and internal walls (without wind loading) and shear wall design (Figure 2.1).

Figure 2.1. Situations requiring the common forms of loading and calculation checks. (a) Wind-loaded panels. (b) External and internal walls (without wind loading). (c) Shear wall design



Design working life

The design working life of the building/structure should be specified. Indicative values for the different categories are given in Table 2.2.

Ultimate limit states

There are two main ultimate limit states to consider as relevant:

- STR, which is described as 'internal failure or excessive deformation of the structure or structural members, including footings, piles, basement walls, etc., where the strength of construction materials of the structure governs';

Table 2.2. Indicative design working life

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	15–30	Agricultural and similar structures
4	50	Building structures and other common structures
5	100	Monumental building structures, bridges and other civil engineering structures

Data from EN 1990.

- EQU, which is described as '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, and
  – the strengths of construction materials or ground are generally not governing'.

There is also the need to consider serviceability. This is related to deflection and cracking. In most unreinforced masonry, serviceability is subservient to the ultimate check since, normally, the onset of cracking suggests ultimate load has been reached. The exception to this is reinforced masonry, where deflection checks may be very important.

Verifications of static equilibrium and resistance

- (STR): when considering a limit state of rupture or excessive deformation of a section, member or connection (STR), it shall be verified that (EN 1990)

$$E_d \leq R_d \tag{2.1}$$

Eqn 6.8

where

E_d is the design value of the effect of actions such as internal force, moment or a vector representing several internal forces or moments
R_d is the design value of the corresponding resistance.

- (EQU): when considering a limit state of static equilibrium of the structure (EQU), it shall be verified that (EN 1990)

$$E_{d,dst} \leq E_{d,stb} \tag{2.2}$$

Eqn 6.7

where

$E_{d,dst}$ is the design value of the effect of destabilising actions
$E_{d,stb}$ is the design value of the effect of stabilising actions.

The loads to consider are detailed in Table 2.3.

With these terms now defined, we can explore the loading side of the equations and determine how E_d is calculated.

EN 1990 gives three basic equations (Equation 2.3) or, alternatively for STR and GEO at the limit states, the less favourable of Equations 2.3a and 2.3b (EN 1990):

$$\sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \tag{2.3}$$

Eqn 6.10

Table 2.3. Types of action

Action type	Symbol	Definition	Common situations
Permanent	G_k	Permanent, invariable load/s during the life of the structure	Self-weight of structures and fixed equipment
Variable	Q_k	Load/s which will or may vary during the life of the structure	Imposed loads on building floors, beams and roofs, wind actions or snow loads
Accidental	A_k	Non normal load/s	Explosions or impact from vehicles or other objects

Eqn 6.10a

$$\left\{ \sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} \psi_{0,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \right. \tag{2.3a}$$

Eqn 6.10b

$$\left\{ \sum_{j \geq 1} \xi_j \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \right. \tag{2.3b}$$

where

$+$ implies 'be combined with'
$\sum$ implies 'the combined effect of'
ξ is a reduction factor for unfavourable permanent actions G.

There are three types of load factors:

γ_f is the partial factor for loads
ψ_0 is a factor for the combination value of a variable action
χ is a reduction factor.

The purpose of the above arrangements is to recognise that, on some occasion in future time when the worst snow load occurs *at the same time* as the worst wind load, it is unlikely that the worst imposed load is also present. Put another way, it is unlikely that when every flat in the block has a Christmas party *on the same evening* in December, both the snow load and the wind load are also acting at their maximum possible values. The probability of all these events happening together is really very small. Taking the analogy further, it is an even smaller probability that, on the day when all these loadings occur simultaneously, there is either a vehicular impact to a ground wall or a gas explosion in one of the flats. The factors ψ_0, ψ_1 and ψ_2 are meant to address these probability scenarios.

For the design of normal superstructures in masonry, Equations 2.3, 2.3a and 2.3b can be thought of as shown in Tables 2.4 and 2.5.

Eqn 6.10

Table 2.4. Design values of actions (STR): Equation 2.3

Persistent and transient design situations	Permanent actions		Leading variable action	Accompanying variable actions	
	Unfavourable	Favourable		Main (if any)	Others
Equation 2.3 (Equation 6.10)	$\gamma_{Gj,sup} G_{kj,sup}$ (1.35)	$\gamma_{Gj,inf} G_{kj,inf}$ (1.00)	$\gamma_{Q,1} Q_{k,1}$ (1.5)		$\gamma_{Q,l} \psi_{0,i} Q_{k,i}$ (1.5)

Data from EN 1990.

Eqn 6.10a
Eqn 6.10b

Table 2.5. Design values of actions (STR): Equations 2.3a and 2.3b

Persistent and transient design situations	Permanent actions		Leading variable action	Accompanying variable actions	
	Unfavourable	Favourable		Main	Others
Equation 2.3a (Equation 6.10a)	$\gamma_{Gj,\mathrm{sup}}G_{kj,\mathrm{sup}}$ (1.35)	$\gamma_{Gj,\mathrm{inf}}G_{kj,\mathrm{inf}}$ (1.0)		$\gamma_{Q,1}\psi_{0,1}Q_{k,1}$ (1.5)	$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$ (1.5)
Equation 2.3b (Equation 6.10b)	$\xi\gamma_{Gj,\mathrm{sup}}G_{kj,\mathrm{sup}}$ $0.925\times(1.35)$	$\gamma_{Gj,\mathrm{inf}}G_{kj,\mathrm{inf}}$ (1.0)	$\gamma_{Q,1}Q_{k,1}$ (1.5)		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$ (1.5)

Data from EN 1990.

Set C (Tables 2.6 and 2.7) is not really applicable to normal superstructure design but may be relevant, for example, should a designer be considering the use of an inverted masonry arch as a foundation element in a building.

Table 2.6. Design values of actions (STR/GEO)

Persistent and transient design situations	Permanent actions		Leading variable action	Accompanying variable actions	
	Unfavourable	Favourable		Main (if any)	Others
Equation 2.3 (Equation 6.10)	$\gamma_{Gj,\mathrm{sup}}G_{kj,\mathrm{sup}}$ (1.0)	$\gamma_{Gj,\mathrm{inf}}G_{kj,\mathrm{inf}}$ (1.0)	$\gamma_{Q,1}\ Q_{k,1}$ (1.3) (0 when favourable)		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$ (1.3) (0 when favourable)

Data from EN 1990.

Table 2.7. Design values of actions (EQU)

Persistent and transient design situations	Permanent actions		Leading variable action	Accompanying variable actions	
	Unfavourable	Favourable		Main (if any)	Others
Equation 2.3 (Equation 6.10)	$\gamma_{Gj,\mathrm{sup}}G_{kj,\mathrm{sup}}$ (1.10)	$\gamma_{Gj,\mathrm{inf}}G_{kj,\mathrm{inf}}$ (0.90)	$\gamma_{Q,1}\ Q_{k,1}$ (1.5) (0 if favourable)		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$ (1.5) (0 if favourable)

The values of γ_f, ψ_0, χ need to be established next: they are taken from the National Annex to BS EN 1990-1-1.

The main values of γ_f used are:

for G_k γ_f is taken as 1.35 (or 1.0 if favourable)
for Q_k γ_f is taken as 1.5 (or 1.0 if favourable).

ψ_0 factors for buildings are shown in Table 2.8, taken from Table NA.A1.1 in the National Annex to BS EN 1990.

As can be seen by inspection, generally speaking:

$\psi_0 = 0.7$ (except for storage areas when $\psi_0 = 1$)
$\psi_0 = 0.5$ for wind loads on buildings.

Table 2.8. Values of 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 $\leq$ 30 kN	0.7	0.7	0.6
Category G: traffic area, 30 kN $<$ vehicle weight $\leq$ 16 0kN	0.7	0.5	0.3
Category H: roofs[a]	0.7	0	0
Snow loads on buildings (see EN 1991-3)			
■ for sites located at altitude $H > 1000$ m a.s.l.	0.70	0.50	0.20
■ for sites located at altitude $H \leq 1000$ m a.s.l.	0.50	0.20	0
Wind loads on buildings (see EN 1991-1-4)	0.5	0.2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0.6	0.5	0

[a] See also EN 1991-1-1, clause 3.3.2(1).

If the form of previous tables is developed for the possible load combinations, the following tables are developed. When thinking about masonry they may help in design.

Note that the snow loading is not dealt with here: it needs to be considered as a separate case.

When considering dead, imposed and wind loads together and taking the normal/standard combinations, the load will be as shown in Tables 2.9–2.11.

Eqn 6.10

Table 2.9. Equation 2.3 (values in bold type represent the leading variable action)

	G_k	Q_k	$Wind_k$
Dead + imposed	1.35	1.5	
Dead + wind	1.35		1.5
Dead + imposed + wind	1.35	**1.5**	0.75[a]
Dead + imposed + wind	1.35	1.05[b]	**1.5**

[a] $= 0.5 \times 1.5$.
[b] $= 0.7 \times 1.5$.

Eqn 6.10a

Table 2.10. Equation 2.3a

	G_k	Q_k	$Wind_k$
Dead + imposed	1.35	1.05[b]	
Dead + wind	1.35		0.75[a]
Dead + imposed + wind	1.35	1.05[b]	0.75[a]

[a] $= 0.5 \times 1.5$.
[b] $= 0.7 \times 1.5$.

Table 2.11. Equation 2.3b (value in bold type represents the leading variable action)

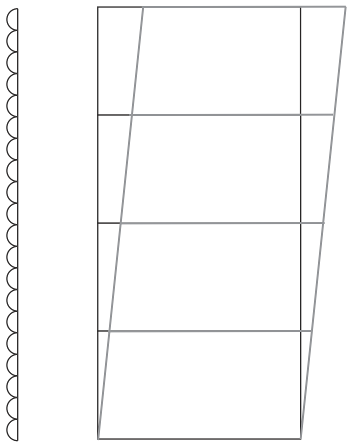
Eqn 6.10b

	G_k	Q_k	Wind_k
Dead + imposed	1.25^a	1.5	
Dead + wind	1.25^a		0.75^b
Dead + imposed + wind	1.25^a	**1.5**	0.75^b
Dead + imposed + wind	1.25^a	1.05^c	**1.5**

[a] = 0.925 × 1.35.
[b] = 0.5 × 1.5.
[c] = 0.7 × 1.5.

Exceptions are shown in Figure 2.2.

Figure 2.2. Shear wall



	G_k	Q_k	Wind_k
'Compression' side			
Worst of either:			
Dead + imposed + wind	1.35	1.5	0.75^a
or			
Dead + imposed + wind	1.35	1.05^b	1.5
'Tension' side			
Dead + wind	1.0		1.5
Check that a compressive stress is present			

[a] = 0.5 × 1.5
[b] = 0.7 × 1.5

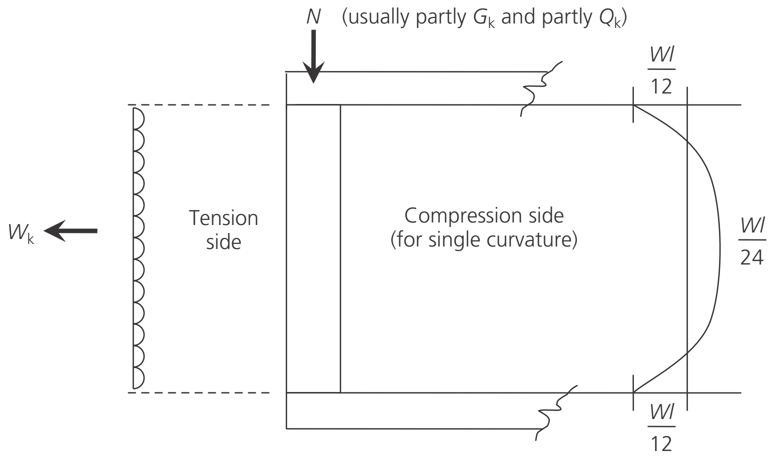
When checking the 'tension' side of the in-plane shear wall, the factors given above for G_k and Wind_k are for the strength condition; that is, they are checking the strength requirements of the wall to assess that there is some compression in the masonry at the extreme fibre.

Were an equilibrium check being conducted on a member, the value of γ_f for G_k should be taken as 0.9 as suggested in Table NA.A.1.2(A) in the National Annex to BS EN 1990.

Figure 2.3 illustrates the wind load check on a load-bearing wall. The worst 'additive' condition is when there is maximum suction on the panel. Note that γ_f for G_k should *not* be taken as 0.9 since this is a strength case and not an equilibrium case.

Figure 2.4 shows an internal load-bearing wall.

Figure 2.3. Wind load check on load-bearing wall



	G_k	Q_k	$Wind_k$
'Compression' side			
Worst of either:			
Dead + imposed + wind	1.35	**1.5**	0.75[a]
or			
Dead + imposed + wind	1.35	1.05[b]	**1.5**
'Tension' side			
Dead + wind	1.0		1.5

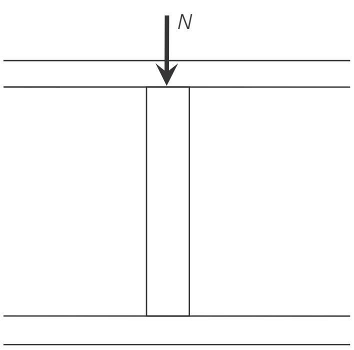
[a] = 0.5 × 1.5
[b] = 0.7 × 1.5

Figure 2.5 illustrates the wind load check on non-load-bearing cladding panel.

The above have introduced the main loading variables and the concept of exploring worst combinations to be used in design calculations.

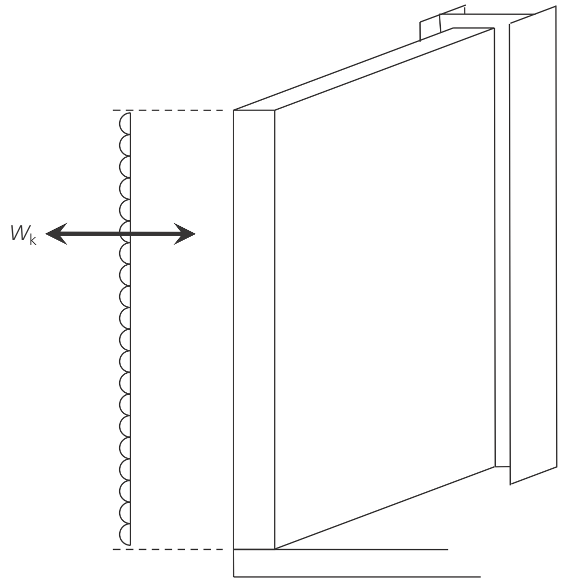
All loading values come from EN 1991. There are several parts to this actions code ('actions' being the new word for 'loading').

Figure 2.4. Internal load-bearing wall



	G_k	Q_k	$Wind_k$
'Compression' side			
Dead + imposed	1.35	1.5	
All over (no pattern loading)			

Figure 2.5. Wind load check on non-load-bearing cladding panel



	G_k	Q_k	$Wind_k$
Flexural tension			
Dead + wind	1.35		1.5

BS EN 1991-1-1:2002	Eurocode 1. Actions on structures. General actions. Densities, self-weight, imposed loads for buildings
BS EN 1991-1-2:2002	Eurocode 1. Actions on structures. General actions. Actions on structures exposed to fire
BS EN 1991-1-3:2003	Eurocode 1. Actions on structures. General actions. Snow loads
BS EN 1991-1-4:2005	Eurocode 1. Actions on structures. General actions. Wind actions
BS EN 1991-1-5:2003	Eurocode 1. Actions on structures. General actions. Thermal actions
BS EN 1991-1-6:2006	Eurocode 1. Actions on structures. General actions. Actions during execution
BS EN 1991-1-7:2006	Eurocode 1. Actions on structures. General actions. Accidental actions

Some loads, of course, may come from consideration of geotechnics in Eurocode 7.

For masonry, the structural dead loads usually predominate. It is always useful to remember that manufacturers' guidance can prove useful here.

When considering accidental damage, the generic Table 2.12 (Table A1.3 from EN 1990) should be used.

If using BS EN 1996 (i.e. designing in the UK – or using the UK approach), the values from the UK National Annex should be used (Table 2.13), where it can be seen that the UK has adopted ψ_1 for the accidental load combinations.

When considering accidental damage in the UK, the specific values for the UK National Annex to BS EN 1990 (see Table 2.13) should be used.

Table 2.12. Design values of actions for use in accidental and seismic combinations of actions

Design situation	Permanent actions		Leading accidental or seismic action	Accompanying variable actions[b]	
	Unfavourable	Favourable		Main (if any)	Others
Accidental[a] (Equations 6.11a and 6.11b)	$G_{kj,sup}$	$G_{kj,inf}$	A_d	ψ_{11} or $\psi_{21} Q_{k1}$	$\psi_{2,l}\ Q_{k,i}$
Seismic (Equations 6.12a and 6.12b)	$G_{kj,sup}$	$G_{kj,inf}$	$\gamma_I A_{Ek}$ or A_{Ed}		$\psi_{2,l}\ Q_{k,i}$

Data from EN 1990.
[a] In the case of accidental design situations, the main variable action may be taken with its frequent or, as in seismic combinations of actions, its quasi-permanent values. The choice will be in the National Annex, depending on the accidental action under consideration. See also EN 1991-1-2.
[b] Variable actions are those considered in Table A1.1 in EN 1990.

Table 2.13. Design values of actions for use in accidental and seismic combinations of actions

Design situation	Permanent actions		Leading accidental or seismic action	Accompanying variable actions[b]	
	Unfavourable	Favourable		Main (if any)	Others
Accidental (Equations 6.11a and 6.11b)	$G_{kj,sup}$	$G_{kj,inf}$	A_d	$\psi_{11}\ Q_{k,1}$	$\psi_{2,l}\ Q_{k,i}$
Seismic[a] (Equations 6.12a and 6.12b)	$G_{kj,sup}$	$G_{kj,inf}$	$\gamma_I A_{Ek}$ or A_{Ed}		$\psi_{2,l}\ Q_{k,i}$

Data from EN 1998.
[a] The seismic design situation should be used only when specified by the client. See also Eurocode 8.
[b] Variable actions are those considered in Table NA.A1.1 in EN 1998.

When thinking in terms of justifying a member to carry 34 kN/m^2 in an accidental situation, the partial factor for 34 is taken as 1.

The vertical load, N, providing stability may have a G_k component and a Q_k component:

- γ_f for the G_k component is effectively taken as 1
- γ_f for the Q_k component is effectively taken as 0.5 (since ψ_1 is the NDP choice).

Note that in some other countries, ψ_2 may have been chosen as the NDP value – in which case this will be stated in the National Annex to EN 1990. If so, γ_f for the Q_k component should be taken as 0.3 (since ψ_2 is the NDP choice).

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Chapter 3
Materials

Commentary
Any chapter on materials within a structural design code is, by its very nature, important. In the masonry code, however, the chapter on materials takes on an even greater importance since it introduces changes that are new to British practice.

Change 1: the classification of units
In the past, the 'old-style' British standard specification of units has only dealt with those units that are manufactured and used in the UK. A pan-European code has to deal with all the types of units manufactured in all CEN member countries. Many of these units are not units with which building professionals in the UK are familiar (Figure 3.1). This has of necessity required a new *classification* of units to be agreed. The classification introduces the concept of *groups*. Groups 1, 2, 3 and 4 classify brick and block units with varying degrees of 'voiding'. While a lightly perforated unit will belong to Group 1, a unit with a higher degree of voiding may well belong to Group 2 or Group 3. While Groups 1, 2 and 3 are for units containing vertically orientated perforations or voids, units with horizontally orientated perforations or voids (when the units are laid) are defined as Group 4. This is a major change with which UK designers will require to become familiar.

Figure 3.1. (a) Clay units (essentially bricks) traditionally available in Britain and (b) the wider range of units traditionally available in mainland Europe

Change 2: method of determining the characteristic compressive strength
The other major change introduced in this chapter is the way by which a designer determines the characteristic compressive strength, f_k, of masonry.

A new approach – one might actually use the phrase a *new philosophy* – has been introduced in an attempt to deal logically with masonry made from the many different types of units used within the CEN member countries, as mentioned above.

The new approach works as follows:

- Whatever the mean compressive strength of the masonry unit as determined by test, this strength has to be turned into the *normalised mean compressive strength*. The normalised mean compressive strength represents the material strength that would be derived from testing a 100 mm cube of material.
- When the normalised strength of the unit is known, then the characteristic strength of the masonry, f_k, can be determined by using a formula of the form

$$f_k = K f_b^{0.7} f_m^{0.3}$$

where

K is a constant that depends on the group of the unit and the type of material from which it is manufactured
f_b is the normalised mean compressive strength of the unit
f_m is the mean compressive strength of the mortar.

This is quite a different approach to the one with which British designers have been familiar over the years. In the past, f_k could be read from a table appropriate for the type of unit being used. Now, it simply becomes part of the calculation process. Of course, this makes it more amenable to software programming. For those doing *desktop design*, tables giving values of f_k for the units commonly used in the UK are presented in Annex 1 of this guide, along with graphs.

Additionally, this change also accommodates the fact that the normalised mean compressive strength refers to the air dry condition. (Traditionally, for example, clay bricks tested to BS 3921 were always wet-tested.)

With this short introductory commentary, the various clauses of Section 3 will be looked at in turn.

This chapter is concerned with the many and varied components required not only to form unreinforced and reinforced masonry but also to ensure that they form stable masonry structures:

- *Clause 3.1* Masonry units
- *Clause 3.2* Mortar
- *Clause 3.3* Concrete infill
- *Clause 3.4* Reinforcing steel
- *Clause 3.5* Prestressing steel
- *Clause 3.6* Mechanical properties of masonry
- *Clause 3.7* Deformation properties of masonry
- *Clause 3.8* Ancillary components.

Much of Section 3 of EN 1996-1-1 refers the reader to other Eurocodes. Not withstanding this fact, there are useful comments that can be made about most of the above headings.

Clause 3.1

3.1. Masonry units

Clause 3.1.1

3.1.1 Types and grouping of masonry units

Clause 3.1.1(1)P

For masonry to be designed to EN 1996, the masonry must be constructed using one of six generic types of masonry unit. The six generic types are listed below, together with the relevant EN number that controls their manufacture and specification:

- clay units EN 771-1
- calcium silicate units EN 771-2
- aggregate concrete units (dense and lightweight aggregate) EN 771-3
- autoclaved aerated concrete units EN 771-4
- manufactured stone units EN 771-5
- dimensioned natural stone units. EN 771-6

Units that do not conform to the EN 771 series cannot be used in a design claiming to satisfy the requirements of EN 1996.

Clause 3.1.1(2)

Masonry units are declared as being either Category I or Category II. Category I units are units with a declared compressive strength with a probability of failure to reach that compressive strength not exceeding 5%. This may be determined via the mean or characteristic value. Category II units are units not intended to comply with the level of confidence of Category I units (regarding the declared compressive strength).

Commentary
The terms 'Category I' and 'Category II' can be thought of as analogous to the previously used terms 'Special' and 'Normal' for the category of manufacturing control of structural units. Whether a unit was 'Special' or 'Normal' directly affected the value of γ_m that was appropriate to use in BS 5628. There is similarity here with Category I or Category II units.

Clause 3.1.1(3)

Masonry units belong to a group. Broadly speaking, the group to which a unit belongs depends on two factors:

- the degree of perforations/voiding in the unit
- the orientation of the perforations/voiding.

Table 3.1

Using this approach, units fall into one of four groups: Group 1, 2, 3 or 4. The detailed geometrical requirements for these four groups of masonry units are given in Table 3.1 of EN 1996-1-1. The broad-brushstroke requirements are summarised in Table 3.1. It is noted that normally a manufacturer will state the Group into which his unit falls.

Autoclaved aerated concrete, manufactured stone and dimensioned natural stone units are considered to be Group 1; that is, there are no Group 2, 3 or 4 units available.

3.1.2 Properties of masonry units – compressive strength

Clause 3.1.2

The compressive strength of masonry units to be used in design is defined as the *normalised* mean compressive strength, f_b.

The note in clause 3.1.2 suggests that, in the EN 771 series of standards, the normalised mean compressive strength is either:

- declared by the manufacturer or
- obtained by converting the mean compressive strength by using EN 772-1, Annex A (conversion of the compressive strength of masonry units to the normalised mean compressive strength) – see Annex 2 of this guide.

Table 3.1. Broad geometrical requirements of the four groups of masonry units

Group 1	Solid or with a small percentage of vertical perforations/voiding
Group 2	Vertical perforations – normally a larger percentage of perforations/voiding than Group 1
Group 3	Vertical perforations – normally a larger percentage of perforations/voiding than Groups 1 and 2
Group 4	Horizontal perforations/holes

See Table 3.1 of EN 1996-1-1 for details of the allowable percentages of voiding.

When the manufacturer declares the normalised compressive strength of masonry units as a characteristic strength, this should be converted to the mean equivalent using a factor based on the coefficient of variation of the units.

The standard deviation (SD) is sometimes expressed as a percentage of the mean, in which case it is known as the coefficient of variation. Depending on the distribution assumed, the 95% confidence limit – the characteristic value – is approximately equal to the mean: $1.645 \times$ SD.

It is not fully clear whether manufacturers will declare the normalised mean compressive strength or merely the mean. Furthermore, it is material-dependent. Clay unit manufacturers must declare the mean. They may also choose to declare the normalised mean. For concrete units, the manufacturer may declare the mean or the characteristic value. Again, they may choose to also declare the normalised mean. Most manufacturers are aware that the conversion process requires the shape factor from EN 772, which is not given in the structural Eurocode (see Annex 2 of this guide). This may encourage them to declare the normalised mean strength, since they will be aware that the shape factor table is not readily available to the designer.

Clause 3.2

3.2. Mortar

Clause 3.2 deals with definitions of terminology used for mortars.

Clause 3.2.1

3.2.1 Types of mortar

Mortar is defined according to its application:

- A general-purpose mortar is defined as a mortar for use in masonry joints with a thickness greater than 3 mm and in which only normal-weight aggregates are used. This is the type of mortar commonly seen in UK projects, whether residential or commercial developments.
- A thin-layer mortar – as its name suggests – is for use in joints between 1 and 3 mm in thickness.
- A lightweight masonry mortar is defined as mortar with a dry hardened density below a prescribed figure. It is not a mortar that has been used in the UK.

Mortar is also defined according to its concept:

- Within the band of general-purpose mortars, a *designed mortar* is based on the concept of performance. It is defined as a mortar whose composition and manufacturing method is chosen by the producer in order to achieve specified properties – normally strength.
- A *prescribed mortar* is based on the concept of recipe. It is defined as a mortar made in predetermined proportions. The properties of the mortar can be assumed from the stated proportion of the constituents – again, normally strength.

Any general-purpose mortar may be described as either a designed mortar or a prescribed mortar, depending on whether it is to be governed by *performance* or by *recipe*.

By their very definition, thin-layer and lightweight masonry mortars should be designed mortars.

Mortar is further defined according to the location or its mode of manufacture:

- A factory-made mortar is defined as a mortar batched and mixed in a factory. It may be 'dry mortar' which is ready mixed, only requiring the addition of water, or 'wet mortar' which is supplied ready for use.
- A semi-finished factory-made mortar is defined as a mortar that has its constituents wholly batched in a factory and which may be either mixed in the factory or on site to the manufacturer's specification.
- A site-made mortar is defined as a mortar composed of individual constituents batched and mixed on the building site.

Clause 3.2.1(4)P

The principle is set such that whatever type of mortar is used, it should comply with the relevant EN standard. Clause 3.2.1(4)P calls for mortars to be in accordance with:

Table 3.2. Equivalent mortar mixes: BS 5628 and BS EN 1996

Mortar designation (BS 5628)	Compressive strength class (BS EN 1996)
(i)	M12
(ii)	M4
(iii)	M6
(iv)	M2

- EN 998-2 for factory-made, semi-finished factory-made and pre-mixed lime and sand masonry mortar
- EN 1996-2 for site-made masonry mortar.

3.2.2 Specification of masonry mortar

Clause 3.2.2

Designed mortars are classified by compressive strength. An M5 mortar is a 5 N/mm^2 mortar (described as a mean compressive strength at 28 days).

Prescribed mortars (being defined by their constituent mix proportions, e.g. 1 : 1 : 5 cement : lime : sand by volume) do not have an M value as such – although, of course, they will have an actual strength. Moreover, it is necessary to know the strength of these prescribed mortars before the f_k value of the masonry can be established and structural calculations made.

For comparison, the mortar designations given in BS 5628 can be equated in strength terms to those in Eurocode 6, as shown in Table 3.2.

3.2.3 Properties of mortar

Clause 3.2.3

The principle is established that the compressive strength of masonry mortar (which is a mean value after 28 days) shall be determined from EN 1015-11, 'Methods of test for mortar for masonry – Part 11: Determination of flexural and compressive strength of hardened mortar'.

It is also noted that, where reinforcement is to be incorporated:

$$f_m \begin{cases} \geq 2\,\mathrm{N/mm^2} & \text{for bed-joint reinforcement} \\ \geq 4\,\mathrm{N/mm^2} & \text{for all other reinforced masonry} \end{cases}$$

Commentary

The reader will now be aware that M4 and M2 mortars are equivalent to the old mortar designations (iii) and (iv), respectively. In BS 5628, the designations deemed suitable for reinforced masonry were actually one designation stronger. Those who are more relaxed using the conventional approach and limits as advocated by BS 5628 may wish to think of

$$f_m \begin{cases} \geq 4\,\mathrm{N/mm^2} & \text{for bed-joint reinforcement} \\ \geq 6\,\mathrm{N/mm^2} & \text{for all other reinforced masonry} \end{cases}$$

even though M2 and M4 mortars are deemed satisfactory by the wording in Eurocode 6.

The principle is set that the adhesion between the mortar and the masonry units shall be adequate for the intended use. This is a useful additional clause since it sets down a basic principle that is of great importance to the design and use of masonry – namely the adhesion between the units and the mortar. If bond is not an issue, then work can proceed even in areas where sands produce mortar that does not bond well with the structural units. If bond is an issue, the problem posed by the lack of adhesion requires to be addressed before work can commence.

It must be clearly understood that this clause is concerned with the adhesion of mortar to the units: mortar should never be used if it is not cohesive within itself (i.e. where sand grains are not well cemented to surrounding grains, and the resulting mortar is 'friable' and 'soft').

Clause 3.3
Clause 3.3.1

3.3. Concrete infill

Concrete infill shall be specified in accordance with EN 206 and specified by its characteristic compressive strength f_{ck} as defined in EN 206 (concrete strength class).

Clause 3.3.2

The concrete strength class shall not be less than 12/15 N/mm^2. The mix may be designed or prescribed but must contain only sufficient water to balance the requirement of strength with adequate workability.

The important principle is established that the workability should be such that all voids will be completely filled when the concrete is placed with good workmanship in accordance with EN 1996-2.

Guidance is given on the slump classes or flow classes that should prove satisfactory for the majority of cases: greater care (and a higher slump class) is recommended when the smallest dimension of a hole/void is $<$85 mm.

For large aggregate, a maximum size of 20 mm is suggested: this size should be reduced to 10 mm when either:

- the least dimension of a void is $<$100 mm or
- the cover of the reinforcement is $<$25 mm.

Clause 3.3.3

The characteristic compressive strength and the characteristic shear strength of concrete can be obtained from tests. These could be from test data already existing or from special tests conducted for the project.

Where such test data are not available, a fallback table of default values is provided in Table 3.2 of EN 1996-1-1.

Clause 3.4

3.4. Reinforcing steel

This general clause refers the steel specification to EN 10080 and introduces the concept of carbon steel or stainless steel, which is covered by EN 10088. Stainless steel is now the preferred specification for most designers using bed-joint reinforcement in external leaves. Reference is made to the concrete Eurocode (EN 1992-1-1) for detailed information on the properties of reinforcing steel.

The characteristic strength of the reinforcing bars, f_{yk}, is to be in accordance with Annex C of EN 1992-1-1.

Bed-joint reinforcement (stainless or galvanised steel) is to be in accordance with EN 845-3.

Clause 3.5
Clause 3.4.1
Clause 3.4.2
Clause 3.4.3
Clause 3.6

3.5. Prestressing steel

Clause 3.5 refers the reader to EN 10138 (or an appropriate European Technical Approval) and, for the properties, to EN 1992-1-1.

3.6. Mechanical properties of masonry

Commentary

Having outlined earlier the fundamental difference in the way that Eurocode 6 treats the characteristic compressive strength of masonry, there is still one other final nuance. The code permits each member country to either establish the value of f_k from its own database of results or to follow detailed formulae – including default values for K. This would therefore permit a country such as the UK, which has many hundreds of wall test results, to base its value of f_k on these results. This could be done using a table or sets of tables as provided in BS 5628; alternatively, it could also take the form of the equations mentioned earlier – amended to give a best fit to the test data. Of course, a country with no wall results or only a few would have no real alternative to adopting the formulae within Eurocode 6 and using the default values for K.

With that introduction, we now examine the various clauses. The non-UK position – the default position – will be looked at first: only after doing this will we consider the UK National Annex approach.

3.6.1 Characteristic compressive strength of masonry *Clause 3.6.1*

3.6.1.1 General *Clause 3.6.1.1*

The characteristic compressive strength, f_k, of all masonry is to be derived from tests. Such tests can be from a national or regional database as well as from tests associated with a particular project.

3.6.1.2 Characteristic compressive strength of masonry other than shell-bedded masonry *Clause 3.6.1.2*

Clause 3.6.1.2(1)

A choice of approaches is offered, based either on the database approach of clause 3.6.1.2(1)(i) or on the calculation approach given in clause 3.6.1.2(1)(ii). If option (i) is chosen by a member country, f_k values could be given perhaps in a table or generated using Equation 3.1 of EN 1996-1-1, which is given below in generic form:

$$f_k = K f_b^{\alpha} f_m^{\beta} \qquad (3.1)$$ *Eqn 3.1*

where

f_k is the characteristic compressive strength (N/mm^2)
K is a constant
α, β are constants
f_b is the normalised mean compressive strength in the direction of the applied action (N/mm^2)
f_m is the compressive strength of the mortar (N/mm^2).

The choice of which route a country has chosen will be stated in its National Annex.

If option (ii) is chosen, the value for f_k comes from a set of basic design equations. *Eqns 3.2–3.4*

If option (i) is chosen, the method of determining the value of f_k will be given in the National Annex – presumably expressed either in terms of the basic generic equation or as a set of characteristic strength tables.

It is perhaps easier to follow the logic if we consider option (ii) first. These are a set of equations from which a value for f_k is calculated.

For masonry constructed from general-purpose mortar and lightweight mortar, Equation 3.2 from EN 1996-1-1 is used:

$$f_k = K f_b^{0.7} f_m^{0.3} \qquad (3.2)$$ *Eqn 3.2*

For thin-layered mortars, one of the two equations below is appropriate (Equations 3.3 and 3.4, respectively, from EN 1996-1-1):

$$f_k = K f_b^{0.85} \qquad (3.3)$$ *Eqn 3.3*

$$f_k = K f_b^{0.7} \qquad (3.4)$$ *Eqn 3.4*

where

f_k is the characteristic compressive strength (N/mm^2)
K is a constant
f_b is the normalised mean compressive strength in the direction of the applied action (N/mm^2)
f_m is the compressive strength of the mortar (N/mm^2).

Table 3.3. General-purpose mortars: tabulated values for Equation 3.2 of EN 1996-1-1 with $K = 1$

Normalised mean compressive strength of the units: N/mm^2	Mortar classification		
	M12	M6	M4
5	6.2	5.3	4.7
10	10.6	8.6	7.6
15	14.0	11.4	10.1
20	17.2	13.9	12.3
25	20.1	16.3	14.4
30	22.8	18.5	16.4
35	25.4	20.6	18.3
40	27.9	22.6	20.0
45	30.3	24.6	21.8
50	32.6	26.5	23.4
55	34.8	28.3	25.1
60	37.0	30.1	26.6
65	39.2	31.8	28.2
70	41.2	33.5	29.7
≥75	43.3	35.2	31.1

Other requirements need to satisfied. For example, all masonry should be well detailed and conform to Section 8 of EN 1996-1-1 ('Detailing') and, in particular, all joints should be made in such a way that they are considered to be filled. It is also assumed that the wall thickness is the width or the length of the unit, so that there is no mortar joint parallel to the face of the wall through all or any part of the length of the wall. If this does not apply (e.g. a 215-brick wall or a collar-jointed wall), then the appropriate value of K should be reduced by 20% (i.e. $K = 0.8K$) when general-purpose mortar is used.

When general-purpose mortar is used,

$$f_b \leq 75\ \text{N/mm}^2$$

and

$$f_m \leq 20\ \text{N/mm}^2 \qquad \text{and} \qquad f_m \leq 2 f_b$$

When the above rules are followed the value of f_k, the characteristic compressive strength of masonry when using general-purpose mortar, is as shown in Table 3.3 and Figure 3.2 for one value of $K = 1$.

Figure 3.2. General-purpose mortars: characteristic compressive strength of masonry f_k from Equation 3.2 of EN 1996-1-1 with $K = 1$

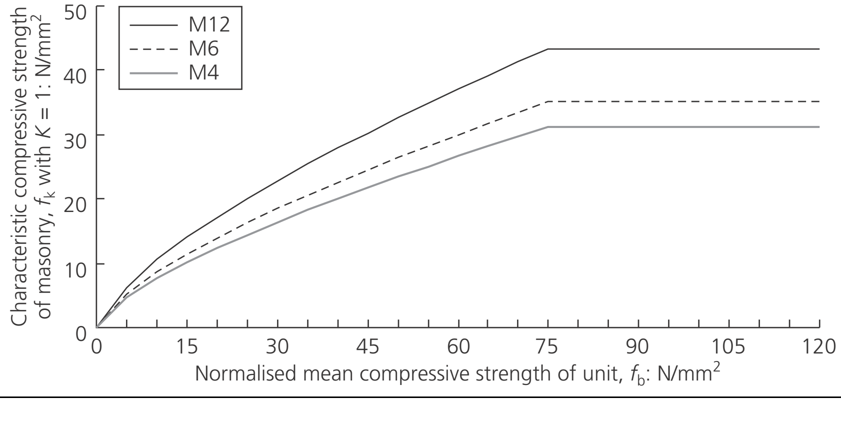

Table 3.4. Lightweight mortars: tabulated values for Equation 3.2 of EN 1996-1-1 with $K = 1$

Normalised mean compressive strength of the units: N/mm^2	Mortar classification		
	M12 (restricted to 10 N/mm^2)	M6	M4
5	6.2	5.3	4.7
10	10.6	8.6	7.6
15	13.3	11.4	10.1
20	16.2	13.9	12.3
25	19.0	16.3	14.4
30	21.6	18.5	16.4
35	24.0	20.6	18.3
40	26.4	22.6	20.0
45	28.7	24.6	21.8
50	30.9	26.5	23.4
55	33.0	28.3	25.1
60	35.1	30.1	26.6
65	37.1	31.8	28.2
70	39.0	33.5	29.7
≥75	41.0	35.2	31.1

When lightweight mortar is used, a different set of sub-rules apply. Now,

$$f_m \leq 10\ \text{N/mm}^2$$

while, for thin-layer mortars, the maximum strength of the unit is reduced to

$$f_b \leq 50\ \text{N/mm}^2$$

The maximum strength of the unit does not appear to be reduced for lightweight mortars. Whether this is deliberate or an oversight by the committee is not clear to the writer.

When the above rules are followed the value of f_k, the characteristic compressive strength of masonry when using lightweight mortar, is as shown in Table 3.4 and Figure 3.3 for $K = 1$. The results are identical to the previous general-purpose mortar table and graph except for the M12 column, which now has to be restricted to 12 N/mm^2.

Figure 3.3. Lightweight mortars: characteristic compressive strength of masonry f_k from Equation 3.2 of EN 1996-1-1 with $K = 1$

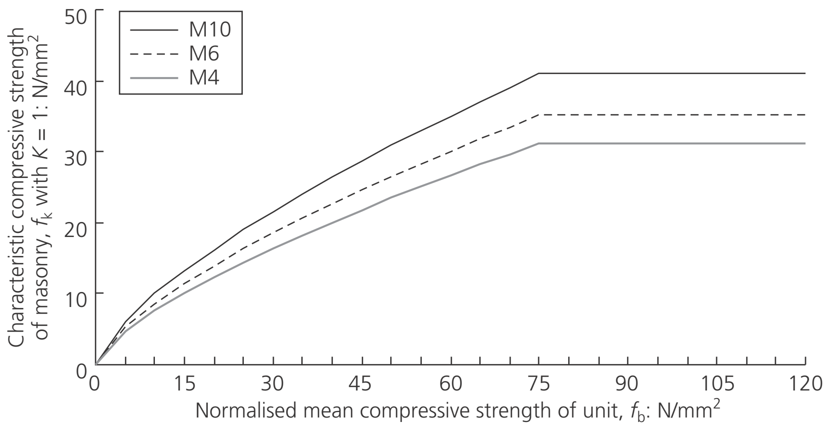

Table 3.5. Thin layer mortar: tabulated values for Equations 3.3 and 3.4 of EN 1996-1-1 with $K = 1$

Normalised mean compressive strength of the units: N/mm^2	Mortar classification	
	Equation 3.3	Equation 3.4
5	3.9	3.1
10	7.1	5.0
15	10.0	6.7
20	12.8	8.1
25	15.4	9.5
30	18.0	10.8
35	20.5	12.0
40	23.0	13.2
45	25.4	14.4
50	27.8	15.5
55	27.8	15.5
≥60	27.8	15.5

When thin-layer mortar is used, a different set of rules apply. Now, either Equation 3.3 or 3.4 of EN 1996-1-1 applies depending on the unit to be used. Note that the mortar strength does not come into the equation: it is the compressive strength of the unit alone that influences.

Eqn 3.3

$$f_k = K f_b^{0.85} \tag{3.3}$$

Eqn 3.4

$$f_k = K f_b^{0.7} \tag{3.4}$$

Equation 3.3 applies for all units other than clay units of Groups 2 and 3. Equation 3.4 should be used when Group 2 and 3 clay units are used.

When thin-layer mortar is used, a different set of limits applies compared with those applied to general-purpose or lightweight mortar. Now,

$$f_b \leq 50 \text{ N/mm}^2$$

When the above rules are followed the value of f_k, the characteristic compressive strength of masonry when using thin-layer mortar, is as shown in Table 3.5 and Figure 3.4.

In some cases (e.g. reinforced masonry beams), the action effects may be parallel to (not perpendicular to) the direction of the bed joints. To apply the above formulae, f_b should be

Figure 3.4. Thin-layer mortar: characteristic compressive strength of masonry f_k from Equations 3.3 and 3.4 of EN 1996-1-1 with $K = 1$

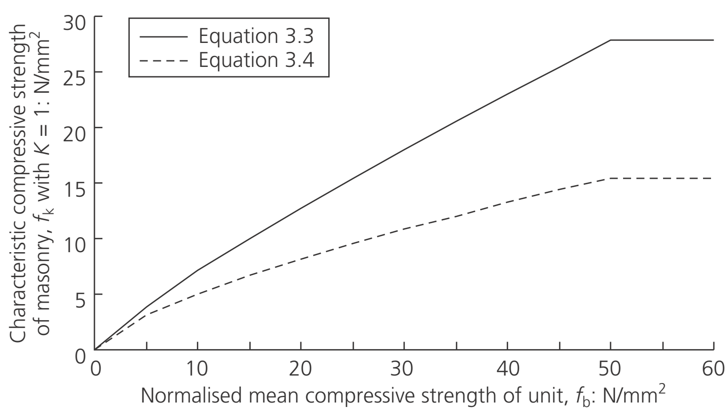

Table 3.6. Values of K for use with general-purpose, thin-layer and lightweight mortars *Table 3.3*

Masonry unit		General-purpose mortar	Thin-layer mortar (bed joint $\geq$0.5 mm and $\leq$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	0.55	0.75	0.30	0.40
	Group 2	0.45	0.70	0.25	0.30
	Group 3	0.35	0.50	0.20	0.25
	Group 4	0.35	0.35	0.20	0.25
Calcium silicate	Group 1	0.55	0.80	‡	‡
	Group 2	0.45	0.65	‡	‡
Aggregate concrete	Group 1	0.55	0.80	0.45	0.45
	Group 2	0.45	0.65	0.45	0.45
	Group 3	0.40	0.50	‡	‡
	Group 4	0.35	‡	‡	‡
Autoclaved aerated concrete	Group 1	0.55	0.80	0.45	0.45
Manufactured stone	Group 1	0.45	0.75	‡	‡
Dimensioned natural stone	Group 1	0.45	‡	‡	‡

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‡ Combination of mortar/unit not normally used, so no value given.

based on tests prototypically in the same direction as the actions in relation to the unit. Note that the shape factor δ (EN 771-1) must not be greater 1. For Group 2 and 3 units, the appropriate value of K should be reduced by 50% (i.e. $K = 0.5K$).

When Group 2 and 3 aggregate concrete units are used *and* where the vertical cavities are completely filled with mortar, f_b should be obtained by considering the unit to be a Group 1 unit with a compressive strength of the lower of either the unit or the concrete infill.

The importance of having fully filled bed joints has already been mentioned. There is, however, little effect on the compressive strength of a wall if all of the perpends are left empty of mortar. It should be recognised that other properties such as sound transmission and rain penetration will usually be very adversely affected by empty perpends.

Finally, Table 3.6 (Table 3.3 of EN 1996-1-1) gives the 'default' values of K for the various masonry units and for general-purpose, thin-layer and lightweight mortars.

Remembering that we are still considering option (ii) of clause 3.6.1.2(1), the following summary of the basic and extra requirements may prove useful to the reader:

$$f_k = Kf_b^{0.7}f_m^{0.3} \tag{3.2}$$ *Eqn 3.2*

for general-purpose and lightweight mortars;

$$f_k = Kf_b^{0.85} \tag{3.3}$$ *Eqn 3.3*

for thin-layer mortars (thickness of 0.5–3 mm) for all units *except* clay bricks of Group 2 and 3; and

$$f_k = Kf_b^{0.7} \tag{3.4}$$ *Eqn 3.4*

for thin-layer mortars (thickness of 0.5–3 mm) for clay brick units of Group 2 and 3.

$$f_b \begin{cases} \leq 75\,\mathrm{N/mm^2} \text{ for general-purpose mortar and lightweight mortars} \\ \leq 50\,\mathrm{N/mm^2} \text{ for thin-layer mortar} \end{cases}$$

$$f_m \begin{cases} \leq 2f_b \text{ for general-purpose mortar} \\ \leq 20\,\text{N/mm}^2 \text{for general-purpose mortar} \\ \leq 10\,\text{N/mm}^2 \text{for lightweight mortar} \end{cases}$$

The tables and graphs above give the values of f_k for a range of values of f_b for a value of K of unity.

If a country opts for the option (ii) route of clause 3.6.1.2(1), it will be clearly stated in the National Annex: the design approach discussed above will then apply *in all and every* detail. If, however, a country opts for the option (i) approach of the clause, the above approach should not be used and the detailed approach given in the National Annex should be used. This will vary from country to country but, to help the reader, the approach adopted by the UK will be discussed at the end of this chapter.

Clause 3.6.1.3

3.6.1.3 Characteristic compressive strength of shell-bedded masonry

Rules are given for the particular case of shell-bedded masonry. The rules are based on the three basic equations – Equations 3.2, 3.3 and 3.4 of EN 1996-1-1 – discussed above. The rules are both specific and clear, and no further commentary is required. The reader should bear in mind that the view of the manufacturer can always be sought: it is likely that information about the characteristic compressive strength of masonry used with their products will be made available by the various manufacturers.

Clause 3.6.2

3.6.2 Characteristic shear strength of masonry

As is customary, the characteristic shear strength, f_k, of all masonry is to be derived from tests. Such tests can be from a national or regional database as well as from tests associated with a particular project.

EN 1052-3 and EN 1053-4 are referenced as the relevant Eurocodes for the testing regime.

For general-purpose mortars, thin-layer mortars or lightweight mortars where all joints – including perpends – are considered as filled, the basic equation for the characteristic shear strength of masonry is

Eqn 3.5

$$f_{vk} = f_{vko} + 0.4\sigma_d \tag{3.5}$$

with f_{vk} not greater than $0.065f_b$ or f_{vlt}. The question of whether to use the limit of $0.065f_b$ or the limit of f_{vlt} is by national choice, and this will be stated in the National Annex.

f_{vko} is the characteristic initial shear strength, under zero compressive stress
f_{vlt} is a limit to the value of f_{vk}
σ_d is the design compressive stress perpendicular to the shear in the member at the level under consideration (this would presumably be an average value over the member being considered)
f_b is the normalised mean compressive strength of the masonry units for the load direction perpendicular to the bed face.

For general-purpose mortars, thin-layer mortars or lightweight mortars where all joints are considered as filled but perpend joints are purposefully left unfilled with the masonry units laid closely abutting each other, the basic equation for the characteristic shear strength of masonry is modified slightly:

Eqn 3.6

$$f_{vk} = 0.5\,f_{vko} + 0.4\gamma_d \tag{3.6}$$

with f_{vk} not greater than $0.045f_b$ or f_{vlt}. (The reader should note that there was a typing error in the code when published, and the line in the Note may read 'not greater than $0.065f_b$ or f_{vlt}'. It is meant to read, 'not greater than $0.045f_b$ or f_{vlt}'.)

Again, the question of whether to use the limit of $0.045f_b$ or the limit of f_{vlt} is by national choice, and will be stated in the National Annex.

Table 3.7. Values of the initial shear strength of masonry, f_{vko}

Table 3.4

Masonry units	f_{vko}: N/mm^2			
	General-purpose mortar of the strength class given		Thin-layer mortar (bed joint ≥0.5 mm and ≤3 mm)	Lightweight mortar
Clay	M10–M20	0.30	0.30	0.15
	M2.5–M9	0.20		
	M1–M2	0.10		7
Calcium silicate	M10–M20	0.20	0.40	0.15
	M2.5–M9	0.15		
	M1–M2	0.10		
Aggregate concrete	M10–M20	0.20	0.30	0.15
Autoclaved aerated concrete	M2.5–M9	0.15		
Manufactured stone and Dimensioned natural stone	M1–M2	0.10		

For shell-bedded masonry with a minimum of two strips of general-purpose mortar (as specified), the basic equation for the characteristic shear strength of masonry is

$$f_{vk} = \frac{g}{t} f_{vko} + 0.4\sigma_d \tag{3.7}$$

f_{vk} should not exceed the value of f_{vk} derived from clause 3.6.2.(4) (the no-mortar-in-the-perpends-case, as above), and

g is the total width of the mortar strips
t is the thickness of the wall.

Table 3.7 (Table 3.4 of EN 1996-1-1) provides a range of 'default' values for f_{vko}, depending on the type of masonry unit and the strength class of the mortar.

Instead of using Table 3.4, f_{vko} can become a nationally determined parameter and may be individually chosen from a database. If it is decided that the values in Table 3.3 are to be used in a country, it will say so in the National Annex. Similarly, values of f_{vko} will be clearly expressed in the National Annex should the decision be made not to adopt the Table 3.4 values.

Note that Table 3.4 is a full-status table and not a table in a Note. Also, to use Table 3.4, all mortars should contain no admixtures or additives.

The vertical shear strength of a junction between two walls can be obtained from either tests or the use of f_{vko}. The wall junction, however, requires to be either fully bonded or fully tied.

3.6.3 Characteristic flexural strength of masonry

Clause 3.6.3

The design of laterally loaded panels follows, in large measure, the method contained in the British code of practice for unreinforced masonry: BS 5628-1.

The two directions of bending strength (weak and the strong direction) are first introduced:

f_{xk1} is the characteristic flexural strength when the plane of failure is parallel to the bed joint – the weak direction of bending
f_{xk2} is the characteristic flexural strength when the plane of failure is perpendicular to the bed joint – the strong direction of bending.

As usual, the principle is set down that the values of flexural strength to be used are to be based on experimental evidence. This may be obtained either from new tests or from a database of available results.

Table 3.5

Table 3.8. Characteristic anchorage strength of reinforcement in confined concrete infill

Strength class of concrete	C12/15	C16/20	C20/25	C25/30 or stronger
f_{bok} for plain carbon steel bars: N/mm^2	1.3	1.5	1.6	1.8
f_{bok} for high-bond carbon and stainless steel bars: N/mm^2	2.4	3.0	3.4	4.1

Data from EN 1996-1-1.

When new tests are conducted, they should be done in accordance with the method given in EN 1052-2.

It should be noted that the two tables giving 'default' values for both f_{xk1} and f_{xk2} are in a Note to this section. They do not have full status.

The values to be used in a country will be given in the country's National Annex.

Note that where national values are not to be given in the National Annex (or are simply not available), the values in the tables (given in the Notes to clause 3.6.3(3)) can be used for mortars of M5 or stronger.

Also, special rules are suggested in the Notes for autoclaved aerated concrete (aac) units laid in thin bed mortar.

Clause 3.6.4

3.6.4 Characteristic anchorage strength of reinforcement

The principle is first set that all values for the characteristic anchorage strength of reinforcement embedded in mortar or concrete shall be obtained from experimental evidence.

Table 3.5

Where test data are not available, values can be taken from Table 3.8 (Table 3.5 of EN 1996-1-1) when:

- the reinforcement is embedded in concrete sections with dimensions greater than or equal to 150 mm or
- when the concrete infill surrounding the reinforcement is confined within the masonry units so that the reinforcement can be considered to be confined.

Clause 3.6.4(3)
Table 3.6

Where the conditions do not apply, or where reinforcement is embedded in mortar, values may be taken from Table 3.9 (Table 3.6 of EN 1996-1-1). There is no national choice available with these values.

Where prefabricated bed-joint reinforcement is used, it is suggested that f_{bok} be determined either:

- by test – done to comply with the requirements set down in EN 846-2 – or
- by bond calculation using only the longitudinal wires.

Table 3.6

Table 3.9. Characteristic anchorage strength of reinforcement in mortar or concrete not confined within masonry units

Strength class of:	Mortar	M2–M5	M5–M9	M10–M14	M15–M19	M20
	Concrete	Not used	C12/15	C16/20	C20/25	C25/30 or stronger
f_{bok} for plain carbon steel bars: N/mm^2		0.5	0.7	1.2	1.4	1.4
f_{bok} for high-bond carbon steel and stainless steel bars: N/mm^2		0.5	1.0	1.5	2.0	3.4

Data from EN 1996-1-1.

Presumably a designer would contact the technical department of the manufacturer. Normally, a rule of thumb 'lap length' is available for routine use of a product.

3.7. Deformation properties of masonry

Clause 3.7
Clause 3.7.1
Clause 3.7.2

While no comment is required on the stress–strain relationship of masonry, the determination of the short-term secant modules should be determined from tests (EN 1052-1). These test results may be available in a database for a particular country.

In the absence of a value as determined by tests, the short-term secant modules of elasticity of masonry may be taken as a constant multiplied by f_k. This is offered in the general form

$$E = K_E f_k$$

The 'default' recommended value in Eurocode 6 is $1000f_k$, but K_E will be decided by national choice and given in the National Annex.

The long-term modulus is determined by

$$E_{\text{long term}} = \frac{E}{1 + \phi_\infty} \qquad (3.8)$$

Eqn 3.8

where ϕ_∞ is the final creep coefficient.

3.7.3 Shear modulus

Clause 3.7.3

No comment is required.

3.7.4 Creep, moisture expansion or shrinkage and thermal expansion

Clause 3.7.4

The principle is first set that all information should be based on experimental evidence.

A table of values is provided that has Note status. It offers a range of values for masonry constructed using the commonly available masonry units. It is not clear to the writer when a designer may call on these values on their own volition. The final creep coefficient is called on, however, in certain circumstances for standard wall design for vertical loads.

The values in the table are by national choice, so it is likely that a country with a reasonable database of values will enter such values as it feels appropriate in its National Annex.

3.8. Ancillary components

Clause 3.8
Clause 3.8.1

Damp-proof courses are defined in performance terms.

Designers familiar with BS 5628 will know that an appendix illustrated some standard strapping details. There is no such content in Eurocode 6. Instead, all ancillary components are defined by a Eurocode standard:

- wall ties — EN 845-1 — *Clause 3.8.2*
- straps hangers and brackets — EN 845-1 — *Clause 3.8.3*
- prefabricated lintels — EN 845-2 — *Clause 3.8.4*
- prestressing devices — EN 1992-1-1 — *Clause 3.8.5*

The UK National Annex

When designing using BS EN 1996-1-1, the following approach is derived from the national choices made in the National Annex.

Mortars

Clause 3.2

The proportions of the prescribed constituents required to provide the stated 'M' values for prescribed masonry mortars are given in Table 3.10 (Table NA.2 of the UK National Annex to BS EN 1996-1-1).

Table NA.2

Table 3.10. 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 : 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–d] Notes a–d are given in the National Annex, where comments are also made on cements, combinations of cements, lime, natural aggregates, admixtures and additions, and colouring pigments.

Clause 3.6.1

Characteristic compressive strength of masonry other than shell bedded

As mentioned previously, the UK has opted for option (i), namely to use its database of results. It has done this while still adopting the use of the generic formulae given by Equation 3.1:

$$f_k = K f_b^{\alpha} f_m^{\beta}$$

A designer or a practice can then construct tables from this approach for the unit and mortar combinations which they commonly use. At the same time, a mathematical model is available for software purposes.

Table NA.4

A central requirement in determining f_k is establishing the value for K. This is given in Table 3.11 (Table NA.4 of the UK National Annex to BS EN 1996-1-1).

Table NA.4

Table 3.11. Values of *K* to be used with Equation 3.1

Masonry unit		General-purpose mortar	Thin-layer mortar (bed joint $\geq$0.5 mm and $\leq$3 mm)	Lightweight mortar of density	
				$600 \leq \rho_d \leq 800\ \text{kg/m}^3$	$800 < \rho_d \leq 1300\ \text{kg/m}^3$
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	[a]	[a]
	Group 2	0.40	0.70	[a]	[a]
Aggregate concrete	Group 1	0.55	0.80	0.45	0.45
	Group 1 units laid flat[b]	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	[a]	[a]
Dimensioned natural stone	Group 1	0.45	[a]	[a]	[a]

[a] These units or the unit–mortar combinations have not traditionally been used in the UK, so no values are available.
[b] See the UK National Annex regarding the reduction in the value of *K* when Group 1 units are voided (multiply *K* by $(100 - n)/100$, where n is the percentage of voids (maximum of 25%)).

The values for α and β are taken from the default values given earlier. For general-purpose and lightweight mortars they are 0.7 and 0.3, respectively. For properly constructed thin-layer mortar, they are either 0.85 or 0.7 for α and 0 for β. (The 0.7 value applies only to Group 2 clay units.) Limitations apply to the maximum strength of the unit and to the maximum strength of the mortar. While the limitations broadly follow the limitations mentioned above for the option (ii) approach, the details of these limitations can be found in the National Annex. Thus, when using the UK National Annex with BS EN 1996-1-1,

$$f_k = Kf_b^{0.7}f_m^{0.3} \tag{3.2}$$ *Eqn 3.2*

for general-purpose and lightweight mortars;

$$f_k = Kf_b^{0.85} \tag{3.3}$$ *Eqn 3.3*

for thin-layer mortars (thickness of 0.5–3 mm) for all units *except* clay bricks of Group 2 (see the exact wording in the National Annex); and

$$f_k = Kf_b^{0.7} \tag{3.4}$$ *Eqn 3.4*

for thin-layer mortars (thickness of 0.5–3 mm) for clay brick units of Group 2.

For masonry made with general-purpose mortar and where the thickness of the masonry is equal to the width or length of the unit, so that there is no mortar joint parallel to the face of the wall through all or any part of the length of the wall, K is obtained from Table 3.11.

There are also changes to some of the limits offered when the above equations are used. These are

$$f_b \begin{cases} \leq 110\,\text{N/mm}^2 \text{ for general-purpose mortar and lightweight mortars} \\ \leq 50\,\text{N/mm}^2 \text{ for thin-layer mortar} \\ \text{The coefficient of variation of the strength of the masonry units is not more than 25\%} \end{cases}$$

$$f_m \begin{cases} \leq 2f_b \\ \leq 12\,\text{N/mm}^2 \text{ for general-purpose mortar} \\ \leq 10\,\text{N/mm}^2 \text{ for lightweight mortar} \end{cases}$$

For masonry made with general-purpose mortar and where there is a mortar joint parallel to the face of the wall through all or any part of the length of the wall, the value of K obtained from Table 3.11 is multiplied by 0.8.

For masonry made of general-purpose mortar where Group 2 and Group 3 aggregate concrete units are used with the vertical cavities filled completely with concrete, the value of f_b should be obtained by considering the units to be Group 1 having a compressive strength corresponding to the compressive strength of the units or of the concrete infill, whichever is the lesser.

Where action effects are parallel to the direction of the bed joints, the characteristic compressive strength may be determined from Equation 3.1 with f_b derived from BS EN 772-1, where the direction of application of the load to the test specimens is in the same direction as the direction of the action effect in the masonry, but with the factor δ (as given in BS EN 772-1) taken to be no greater than 1.0. For Group 2 and 3 units, K should then be multiplied by 0.5.

When the perpend joints are unfilled, the above equations may be used considering also any horizontal actions that might be applied to, or be transmitted by, the masonry (see also clause 3.6.2(4)).

Clause 3.6.1.2(1)(ii)

Commentary

The UK National Annex approach with its own table of K values and its own set of limiting conditions is similar to but different from the default approach given in option (ii) in clause 3.6.1.2(1).

With knowledge of

- the default approach and
- the UK National Annex approach

the reader is now in a position where f_k values can be calculated for any combination of unit and mortar.

Some f_k values are calculated in Annex 3 of this guide – first for the approach given in the UK National Annex and then for the approach that uses the default values given in the code.

Clause 3.6.2

Characteristic shear strength of masonry

The limit of f_{vk} should be taken as $0.065f_b$ for normal masonry, while the limit of f_{vk} should be taken as $0.045f_b$ for masonry with unfilled perpend joints.

In adjusting the UK strength classes for mortar, the UK has adopted the 'default' values recommended by Eurocode 6 for f_{vko} (Table 3.12).

Clause 3.6.3

Characteristic flexural strength of masonry

The values used by the UK in the National Annex are essentially derived from BS 5628 (Table 3.13). This is for two main reasons:

- The BS 5628 values were established from an exhaustive series of tests: they are therefore totally based on experimental evidence.
- There is no reason to alter or adjust them since the design methodology is similar to the design approach in BS 5628.

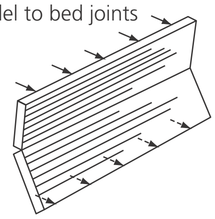
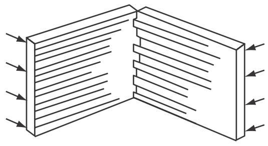
The whole flexural design package is therefore one which UK designers will recognise if they are already familiar with BS 5628.

Table NA.5

Table 3.12. 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 $\leq$0.5 mm and $\geq$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		

Table 3.13. Characteristic flexural strength of masonry, f_{xk1} and f_{xk2} (N/mm^2) *Table NA.6*

	Values of f_{xk1} Plane of failure parallel to bed joints 			Values of f_{xk2} Plane of failure perpendicular to bed joints 		
Mortar strength class:	M12	M6 and M4	M2	M12	M6 and M4	M 2
Clay masonry units of groups 1 and 2 having a water absorption (see note 1) of:						
less than 7%	0.7	0.5	0.4	2.0	1.5	1.2
between 7% and 12%	0.5	0.4	0.35	1.5	1.1	1.0
over 12%	0.4	0.3	0.25	1.1	0.9	0.8
Calcium silicate brick-sized* masonry units	0.3		0.2	0.9		0.6
Aggregate concrete brick-sized* masonry units	0.3		0.2	0.9		0.6
Aggregate concrete masonry units and manufactured stone of groups 1 and 2 and AAC masonry units used in walls of thickness up to 100 mm (see note 4), of declared compressive strength:						
2.9				0.40		0.4
3.6	0.25		0.2	0.45		0.4
7.3				0.60		0.5
Aggregate concrete masonry units and manufactured stone of groups 1 and 2 and AAC masonry units used in walls of thickness of 250 mm or greater (see note 4), of declared compressive strength:						
2.9				0.25		0.2
3.6	0.15		0.1	0.25		0.2
7.3				0.35		0.3
Aggregate concrete masonry units and manufactured stone of groups 1 and 2 and AAC masonry units used in walls of any thickness (see note 2), of declared compressive strength:						
10.4	0.25		0.2	0.75		0.6
≥17.5				0.90 (see note 3)		0.7 (see note 3)

Note 1 Tests to determine the water absorption of clay masonry units are to be conducted in accordance with EN 772-7.

Note 2 The thickness should be taken to be the thickness of the wall for a single-leaf wall, or the thickness of the leaf for a cavity wall.

Note 3 When used with flexural strength in the parallel direction, assume the orthogonal ratio $\mu = 0.3$.

Note 4 Linear interpolation may be used to obtain the values of f_{xk1} and f_{xk2} for:
(a) wall thicknesses greater than 100 mm and less than 250 mm
(b) compressive strengths between 2.9 N/mm^2 and 7.3 N/mm^2 in a wall of given thickness.

* 215 mm × 65 mm × 102.5 mm.

Table 3.14. Values for the final creep coefficient, long-term moisture expansion or shrinkage and the coefficient of thermal expansion for masonry

Type of masonry unit	Final creep coefficient,[a] ϕ_∞	Long-term moisture expansion or shrinkage[b]: mm/m	Coefficient of thermal expansion, α_t: 10^{-6}/K
Clay	1.5	0.5	6
Calcium silicate	1.5	−0.2	10
Dense aggregate concrete and manufactured stone	1.5	−0.2	10
Lightweight aggregate concrete	1.5	−0.4	10
Autoclaved aerated concrete	1.5	−0.2	10
Natural stone	Normally very low	0.1	10

Data from NA to BS EN 1996-1-1:2005.
[a] The final creep coefficient $\phi = \varepsilon_{c\infty}/\varepsilon_{el}$, where $\varepsilon_{c\infty}$ is the final creep strain and $\varepsilon_{el} = \sigma/E$.
[b] Where the long-term value of the moisture expansion or shrinkage is shown as a negative number it indicates shortening, and as a positive number it indicates expansion.

Clause 3.7.2

Modulus of elasticity

The UK has adopted the value of K_E to be 1000, giving $E = 1000f_k$. This is the recommended 'default' value: in the past, UK design practice was to use $900f_k$. It should however be borne in mind that the two values of characteristic strength may not be the same – even for the same unit/mortar combination. The whole basis of design and assessment of masonry strength has been realigned in the new approach used in Eurocode 6.

Clause 3.7.4
Table NA.7

Creep, moisture expansion or shrinkage and thermal expansion

The values offered in Table 3.14 for the final creep coefficient, long-term moisture expansion or shrinkage and the coefficient of thermal expansion for masonry were arrived at after lengthy discussions. They are considered to be fair estimates.

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Chapter 4
Durability

4.1. General

Clause 4.1
Clause 4.1(1)P
Clause 4.2

The principle is first set that masonry should have the required durability for its intended use, taking into account the relevant environmental conditions (clause 4.2 on the classification of environmental conditions).

4.3. Durability of masonry

Clause 4.3

The following clauses then remind the designer that, for the masonry to be durable, the following individual elements require to be durable, namely:

Clause 4.3.1
Clause 4.3.2
Clause 4.3.3
Clause 4.3.4
Clause 4.3.5
Clause 4.3.6

- the masonry units
- the mortar
- any reinforcing or prestressing steel or prestressing devices
- any ancillary components such as frame cramps or shelf angles.

Clause 4.4

A small section about masonry below ground concludes this chapter.

Clause 4.2

The reader is referred to EN 1996-2 for the detailed clauses on durability (see Section 2.1 of EN 1996-2, 'Factors affecting the durability of masonry').

Because durability is important when designing masonry in UK, it is worth spending a little more time on this area than the size of Section 4 of the Eurocode would suggest. The subject needs to be broken down into plain (unreinforced) masonry and reinforcement protection for reinforced masonry.

Clause 4.3.1
Clause 4.3.2

Looking at plain masonry first, the primary requirement is to avoid frost failure of the masonry – both the unit (usually clay brickwork) and the mortar.

The general approach is to avoid saturation by the use of careful details that throw off water. If flush details are required, it will usually involve the masonry becoming very wet or saturated. Such masonry becomes susceptible to frost failure unless frost-resistant bricks and a suitable frost-resistant mortar are specified. The brickwork shown in Figure 4.1 is in a freestanding wall that, depending on the location, can be a more severe exposure situation than the external wall of dwellings, since it can usually be wetted from both sides.

Figures given in Annexes A.2 and A.3 of EN 1996-2 are useful here to help with understanding, particularly the way in which the details can interplay with the micro-climate around a building (Figures 4.2 and 4.3).

EN 771-1 Clay bricks are classified as:

- F0: passive exposure
- F1: moderate exposure
- F2: severe exposure.

By and large, the stronger the mortar, the higher is its durability.

Assuming that we are in the UK, the task now is to find the appropriate brick and mortar combination – the minimum required specification – for a particular position in the building,

Figure 4.1. Example brickwork of a freestanding wall

whatever the location of that building. Guidance on this can be found in Table 15 of PD 6697, entitled 'Durability of masonry in finished construction'. This table gives guidance on the required (minimum) specification for given situations: it deals fully with masonry at or below ground level – so the requirements of clause 4.4 on masonry below ground are also fully covered: see Table 4.1.

Clause 4.4

There are other things that can visually affect masonry – such as efflorescence or lime bloom. These tend not to be issues that affect durability *per se*; that is, they do not directly cause the disintegration of either the unit or the face of the mortar bed.

Sulphate attack, on the other hand, is a very different matter that requires to be taken seriously. It is an attack on the mortar – or one chemical constituent of it. The durability guidance in PD 6697 covers the specification of masonry that is considered to be at risk from sulphates, whether the sulphates come from within the clay bricks themselves or from within the ground where the masonry is to be constructed (see Table 4.1).

Figure 4.2. Examples of the effect of building detail on relative exposure to wetting of masonry: (a) coping with overhang; (b) coping without overhang (simple capping); (c) sill with overhang; (d) sill without overhang (flush sill). (Reproduced from BS EN 1996-2 © British Standards Institution 2006)

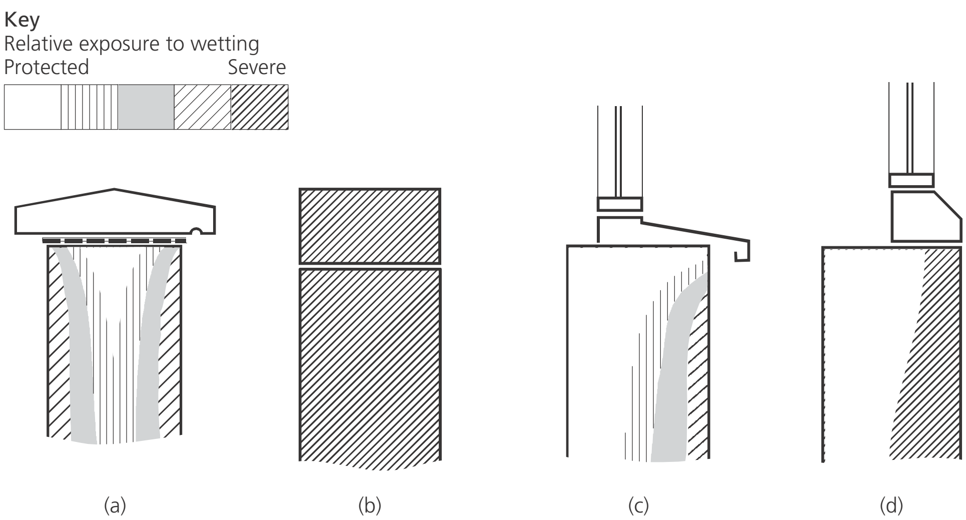
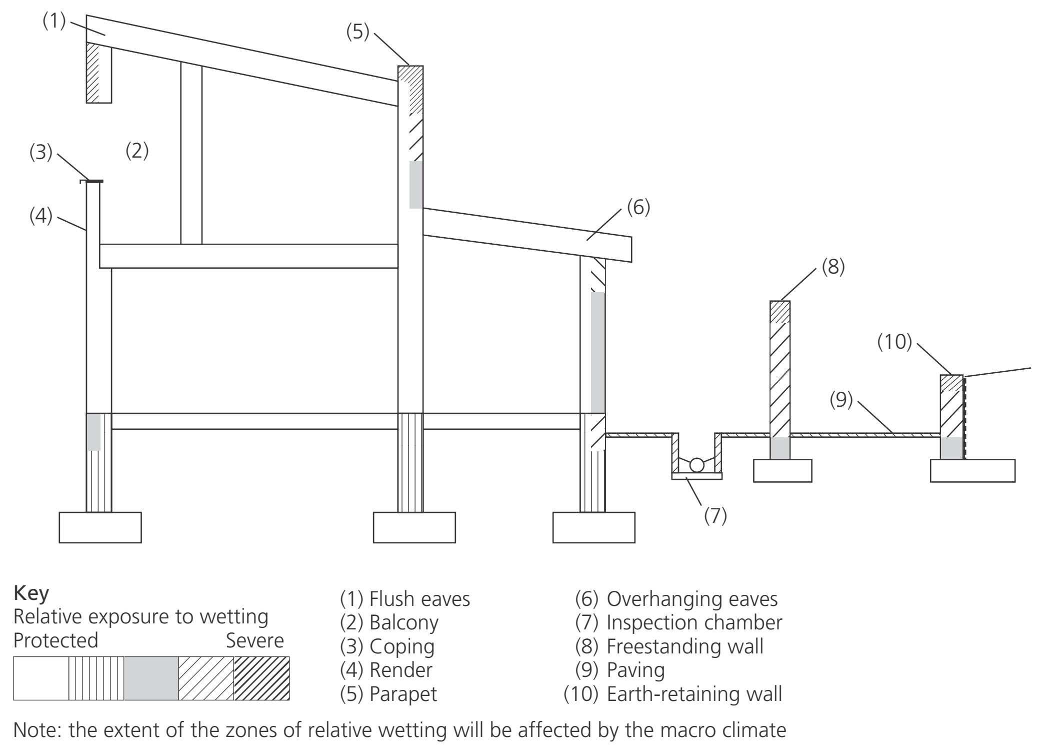


Figure 4.3. Examples of relative exposure to wetting of masonry (not protected by applied finishes or cladding except where indicated, foundation in well-drained soil). (Reproduced from BS EN 1996-2 © British Standards Institution 2006)



Annex A of EN 1996-2 contains a classification for durability. It is shown in Table 4.2, and is entitled 'Classification of micro conditions of exposure of completed masonry'.

The classification extends from MX1 (the friendliest environment for durability – such as internal masonry within a building) through to MX5 (an aggressive environment such as one with significant groundwater sulphate levels).

The increase in severity – from MX1 to MX5 – can be clearly seen by inspection of Table 4.2. It should be noted, however, that the MX2 classification is not one that is prevalent in the UK where freeze/thaw cycling affects most external masonry.

Notwithstanding this, the suggestions for minimum specifications given in Table 4.1 are specifications which are *tried and tested* and known to work in British practice.

There is one recent amendment to Table 4.1 about which the reader should be aware: some manufacturers of clay units do not recommend the use of their HD-F1 units for work below or near external ground level. The reason appears to be that if the soil is not well-draining soil (e.g. if it is stiff clay), saturation of the face of the units can occur and, in winter, this may result in spalling due to the frost action of freeze/thaw cycles.

Figure 4.4(a) shows the effect of just such frost action at the base of a free-standing wall. Had the wall geometry been a straight plate, the loss of thickness due to spalling at (both sides of) the base would reduce the lateral strength of the wall under wind loading. In this particular case, the effect of the spalling is not as significant as it might otherwise have been since the strength of the wall is derived as much from the castellated geometric plan-section (Figure 4.4(b)) as from the width of the wall section itself.

In PD 6697, more guidance is given on exposure in clause 6.2.7 (exclusion of water). It rates the category of exposure to the quantity of rain and the degree by which it is wind driven onto the masonry. It uses the *spell index*, a means of quantifying the degree of exposure to wind-driven rain that is available in the UK.

Table 4.1. Durability of masonry in finished construction

Masonry condition or situation	Quality of masonry units and appropriate mortar designations				Remarks
	Clay units	Calcium silicate units	Aggregate concrete bricks	Aggregate concrete and autoclaved aerated concrete blocks	
(A) Work below or near external ground level					
A1 Low risk of saturation Without freezing MX2.1, MX2.2 With freezing MX3.1, MX3.2	LD-F0 and S0 or HD-F0, F1 or F2 and S0, S1 or S2 in M12, M6 or M4 HD-F1 or F2 and S0, S1 or S2 in M12, M6 or M4 unless a manufacturer advises against the use of HD-F1 (See remarks)	Without or with freezing Compressive strength class 20 or above in M4 or M2 (see remarks)	Without or with freezing Compressive strength 16.5 N/mm^2 or above in M4	Without or with freezing (a) Of net density $\leq$1500 kg/m^3 or (b) made with dense aggregate conforming to BS EN 12620 or (c) having a compressive strength 7.3 N/mm^2 or (d) most types of autoclaved aerated block (see remarks) All in M4 or M2 (see remarks)	Some types of autoclaved aerated concrete block may not be suitable. The manufacturer should be consulted In sulphate-bearing ground conditions, the recommendations in 6.2.8.4 should be followed Where designation M2 mortar is used it is essential to ensure that all masonry units, mortar and masonry under construction are protected fully from saturation and freezing Some manufacturers of clay units do not recommend the use of their HD-F1 units for work below or near external ground level
A2 High risk of saturation without freezing MX2.2	HD-F1 or F2, and S1 and S2 in M12 or M6 (see remarks)	Compressive strength class 20 or above in M6 or M4	Compressive strength 16.5 N/mm^2 or above in M6 or M4	As for A1 in M6 or M4	Masonry most vulnerable in situations A2 and A3 is located between 150 mm above and 150 mm below finished ground level. In this zone, masonry will become wet and can remain wet for long periods, particularly in winter. Where S1 clay units in Designation M6 mortar are used in A2 or A3 locations, sulphate-resisting Portland cement should be used in the mortar (see 6.2.8.4)
A3 High risk of saturation with freezing MX3.2	HD-F2 and S1 or S2 in M12 or M6 (see remarks)	Compressive strength class 20 or above in M6 or M4	Compressive strength 22 N/mm^2 or above in M6 or M4	As for A1 in M6	

						In conditions of highly mobile groundwater, consult the manufacturer on the selection of materials (6.2.8.1.4)
(B) Masonry damp-proof courses (DPCs)						
B1	In buildings MX3.1 MX4	DPC units, maximum water absorption 4.5% in M12	Not suitable	Not suitable	Not suitable	Masonry DPCs can resist rising damp, but will not resist water percolating downwards If sulphate ground conditions exist, the recommendations in 6.2.8.4 should be followed
B2	In external works MX3.1 MX4, MX5	DPC units, max. water absorption 7% in M12	Not suitable	Not suitable	Not suitable	DPCs of clay units are unlikely to be suitable for walls of other masonry units, as differential movement can occur (see 5.4)
(C) Unrendered external walls (other than chimneys, cappings, copings, parapets, sills)						
C1	Low risk of saturation MX3.1, MX4, MX5	HD-F1 or F2 and S1 or S2 in M12, M6 or M4	Compressive strength class 20 or above in M4 or M2 (see remarks)	Compressive strength 7.3 N/mm^2 or above in M4	Any in M4 or M2 (see remarks)	To minimise the risk of saturation, walls should be protected by roof overhang and other projecting features. However, such details may not provide sufficient protection to walls in conditions of very severe driving rain (see 6.2.7.4). Certain architectural features, e.g. brick masonry below large glazed areas with flush sills, increase the risk of saturation (see 6.2.8.5) Where Designation M2 mortar is used it is essential to ensure that all masonry units, mortar and masonry under construction are protected fully from saturation and freezing
C2	High risk of saturation MX3.2 MX4, MX5	HD-F2 and S1 or S2 in M12 or M6 (see remarks)	Compressive strength class 20 or above in M4	Compressive strength 18 N/mm^2 or above in M4	Any in M4	Where S1 clay units are used in Designation M6 mortar for situation C2, sulphate-resisting Portland cement should be used in the mortar (see 6.2.8.4)

Table 4.1. Continued

Masonry condition or situation	Quality of masonry units and appropriate mortar designations				Remarks
	Clay units	Calcium silicate units	Aggregate concrete bricks	Aggregate concrete and autoclaved aerated concrete blocks	
(D) Rendered external walls (other than chimneys, cappings, copings, parapets, sills)					
Rendered external walls	HD-F1 or F2 and S1 or S2 in M12, M6 or M4 (see remarks)	Compressive strength class 20 or above in M4 or M2 (see remarks)	Compressive strength 7.3 N/mm^2 or above in M4	Any in M4 or M2 (see remarks)	Rendered walls are usually suitable for most wind-driven rain conditions (see 6.2.7.4) Where S1 clay units are used, sulphate–resisting Portland cement should be used in the jointing mortar and in the base coat of the render (see 6.2.8.4) Clay units of F1/S1 designation are not recommended for the rendered outer leaf of a cavity wall with full-fill insulation (see 6.2.7.4.2.9) Where Designation M2 mortar is used it is essential to ensure that all masonry units, mortar and masonry under construction are protected fully from saturation and freezing
(E) Internal walls and inner leaves of cavity walls above DPC level					
Internal walls and inner leaves of cavity walls MX1	LD-F0 and S0 or HD-F0, F1 or F2 and S0, S1 or S2 in M12, M6, M4 or M2 (see remarks)	Compressive strength class 20 or above in M4 or M2 (see remarks)	Compressive strength 7.3 N/mm^2 or above in M4 or (iv) (see remarks)	Any in M4 or M2 (see remarks)	Where Designation M2 mortar is used it is essential to ensure that all masonry units, mortar and masonry under construction are protected fully from saturation and freezing (see A.4.1.3.2 and A.5.1.1)

(F) Unrendered parapets (other than cappings and copings)						
F1	Low risk of saturation, e.g. low parapets on some single-storey buildings MX3.1 MX4	HD-F1 or F2 and S1 or S2 in M12, M6 or M4	Compressive strength class 20 or above in M4	Compressive strength 22 N/mm^2 or above in M4	(a) Of net density $\geq$1500 kg/m^3 or (b) made with dense aggregate conforming to BS EN 12620 or (c) having a compressive strength 7.3 N/mm^2 or (d) most types of autoclaved aerated block (see remarks) in M4	Most parapets are likely to be severely exposed irrespective of the climatic exposure of the building as a whole. Copings and DPCs should be provided wherever possible Some types of autoclaved aerated concrete block may not be suitable. The manufacturer should be consulted
R2	High risk of saturation, e.g. where a capping only is provided for the masonry MX3.1, MX3.2 MX4	HD-F2 and S1 or S2 in M12 or M6 (see remarks)	Compressive strength class 20 or above in M4	Compressive strength 22 N/mm^2 or above in M4	As for F1 in M6	Where S1 clay units are used in situation F2, sulphate-resisting Portland cement should be used in the mortar (see 5.6.4)
(G) Rendered parapets (other than cappings and copings)						
	Rendered parapets MX3.1, MX3.2 MX4	HD-F1 or F2 and S2 in M12, M6 or M4 or HD-F1 or F2 and S1 in M12 or M6 (see remarks)	Compressive strength class 20 or above in M4	Compressive strength 7.3 N/mm^2 or above in M4	Any in M4	Single-leaf walls should be rendered only on one face. All parapets should be provided with a coping Where S1 clay units are used, sulphate-resisting Portland cement should be used in the jointing mortar and in the base coat of the render (see 6.2.8.4)
(H) Chimneys						
H1	Unrendered with low risk of saturation MX3.1, MX3.2 MX4, MX5	HD-F1 or F2 and S1 or S2 in M12, M6 or M4 (see remarks)	Compressive strength class 20 or above in M4 (see remarks)	Compressive strength 12 N/mm^2 or above in M4 (see remarks)	Any in M4	Chimney stacks are normally the most exposed masonry on any building Because of the possibility of sulphate attack from flue gases, the use of sulphate-resisting Portland cement in the jointing mortar and in any render is strongly recommended (see 5.6.4)

Table 4.1. Continued

Masonry condition or situation	Quality of masonry units and appropriate mortar designations				Remarks
	Clay units	Calcium silicate units	Aggregate concrete bricks	Aggregate concrete and autoclaved aerated concrete blocks	
H2 Unrendered with high risk of saturation MX3.1, MX3.2 MX4, MX5	HD-F2 and S1 or S2 in M12 or M6 (see remarks)	Compressive strength class 20 or above in M4 (see remarks)	Compressive strength 16.5 N/mm^2 or above in M4 (see remarks)	(a) Of net density $\geq$1500 kg/m^3 or (b) made with dense aggregate conforming to BS EN 12620 or (c) having a compressive strength $\geq$7.3 N/mm^2 or (d) most types of autoclaved aerated block (see remarks) in M6	Brick masonry and tile cappings cannot be relied upon to keep out moisture. The provision of a coping is preferable Some types of autoclaved aerated concrete block may not be suitable for use in situation H2: the manufacturer should be consulted
H3 Rendered MX3.1, MX3.2 MX4, MX5	HD-F1 or F2 and S2 in M12, M6 or M4 or HD-F1 or F2 and S1 in M12 or M6 (see remarks)	Compressive strength class 20 or above in M4 (see remarks)	Compressive strength 7.3 N/mm^2 or above in M4 (see remarks)	Any in M4	
(I) Cappings, copings and sills					
Cappings, copings and sills MX3.1, MX3.2	HD-F2 and S1 or S2 in M12	Compressive strength class 30 or above in M6	Compressive strength 33 N/mm^2 or above in M6	(a) Of net density $\geq$1500 kg/m^3 or (b) made with dense aggregate conforming to BS EN 12620 or (c) having a compressive strength $\geq$7.3 N/mm^2 in M6	Autoclaved aerated concrete blocks are not suitable for use in situation I Where cappings or copings are used for chimney terminals, the use of sulphate-resisting Portland cement in the mortar is strongly recommended (see 6.2.8.4) DPCs for cappings, copings and sills should be bedded in the same mortar as the masonry units

(J) Freestanding boundary and screen walls (other than cappings and copings)						
J1	With coping MX3.1, MX3.2 MX4, MX5	HD-F1 or F2 and S1 in M12 or M6 (see remarks) or HD-F1 or F2 and S2 in M12, M6 or M4	Compressive strength class 20 or above in M4	Compressive strength 16.5 N/mm^2 or above in M4	Any in M4	Masonry in freestanding walls is likely to be severely exposed, irrespective of climatic conditions. Such walls should be protected by a coping wherever possible and DPCs should be provided under the coping units and at the base of the wall (see 6.2.7)
J2	With capping MX3.1, MX3.2 MX4, MX5	HD-F2 and S1 or S2 in M12 or M6 (see remarks)	Compressive strength class 20 or above in M4	Compressive strength 22 N/mm^2 or above in M4	(a) Of net density $\geq$1500 kg/m^3 or (b) made with dense aggregate conforming to BS EN 12620 or (c) having a compressive strength 7.3 N/mm^2 or (d) most types of autoclaved aerated block (see remarks) in M6	Where S1 clay units are used for situation J1 in condition of severe driving rain (see 5.5), the use of sulphate-resisting Portland cement in the mortar is strongly recommended (see 6.2.8.4) Where Designation M6 mortar is used for situation J2, the use of sulphate-resisting Portland cement in the mortar is strongly recommended (see 6.2.8.4) Some types of autoclaved aerated concrete block may also be unsuitable: the manufacturer should be consulted
(K) Earth-retaining walls (other than cappings and copings)						
K1	With waterproofing retaining face and coping MX3.1, MX3.2 MX4	HD-F1 or F2 and S1 or S2 in M12 or M6	Compressive strength class 20 or above in M6 or M4	Compressive strength 16.5 N/mm^2 or above in M6	(a) Of block density $\geq$1500 kg/m^3 or (b) made with dense aggregate conforming to BS EN 12620 or (c) having a compressive strength $\geq$7.3 N/mm^2 or (d) most types of autoclaved aerated block (see remarks) in M4	Because of possible contamination from the ground and saturation by ground waters, in addition to subjection to severe climatic exposure, masonry in retaining walls is particularly prone to frost and sulphate attack. Careful choice of materials in relation to the methods for exclusion of water recommended in 6.2.7 is essential It is strongly recommended that such walls be backfilled with free-draining materials. The provision of an effective coping with a

Table 4.1. Continued

Masonry condition or situation	Quality of masonry units and appropriate mortar designations				Remarks
	Clay units	Calcium silicate units	Aggregate concrete bricks	Aggregate concrete and autoclaved aerated concrete blocks	
K2 With coping or capping but no waterproofing on retaining face MX3.1, MX3.2 MX4, MX5	HD-F2 and S1 or S2 in M12	Compressive strength class 30 or above in M6	Compressive strength 33 N/mm^2 or above in M12 or M6	As for K1 but in M12 or M6 (see remarks)	DPC (see 6.2.7) and waterproofing of the retaining face of the wall (see 5.6.1.4) is desirable Where S1 clay masonry units are used, sulphate-resisting Portland cement may be necessary in the mortar (see 6.2.7.4) Some types of autoclaved aerated concrete block are not suitable for use in situation K1: the manufacturer should be consulted Some aggregate concrete blocks are not suitable for use in situation K2; the manufacturer should be consulted
(L) Drainage and sewage, e.g. inspection chambers, manholes					
L1 Surface water MX3.1, MX3.2 MX5	Engineering bricks or F1 or F2 and S1 or S2 in M12 (see remarks)	Compressive strength class 20 or above in M6 or M4	Compressive strength 22 N/mm^2 or above in M4	(a) Of net density $\geq$1500 kg/m^3 or (b) made with dense aggregate conforming to BS EN 12620 or (c) having a compressive strength $\geq$7.3 N/mm^2 in M4	Where S1 clay units are used sulphate-resisting Portland cement should be used in the mortar If sulphate ground conditions exist, the recommendation in 6.2.7.4 should be followed Some types of autoclaved aerated block are not suitable for use in L1: the manufacturer should be consulted Some types of calcium silicate brick are not suitable for use in L2 or L3: the manufacturer should be consulted

L2	Foul drainage (continuous contact with masonry) MX3.1, MX3.2 MX5	Engineering bricks or F1 or F2 and S1 or S2 in M12 (see remarks)	Compressive strength class 50 or above in M6 (see remarks)	Compressive strength 48 N/mm^2 or above with cement content $\geq$350 kg/m^3 in M12 or M6	Not suitable
L3	Foul drainage (occasional contact with masonry) MX3.1, MX3.2 MX5	Engineering bricks or F1 or F2 and S1 or S2 in M12 (see remarks)	Compressive strength class 20 or above in M6 or M4 (see remarks)	Compressive strength 48 N/mm^2 or above with cement content $\geq$350 kg/m^3 in M12 or M6	Not suitable

Table 4.2. Classification of micro-conditions of exposure of completed masonry

Class	Micro-condition of the masonry	Examples of masonry in this condition
MX1	In a dry environment	Interior of buildings for normal habitation and for offices, including the inner leaf of external cavity walls not likely to become damp. Rendered masonry in exterior walls, not exposed to moderate or severe driving rain, and isolated from damp in adjacent masonry or materials
MX2	Exposed to moisture or wetting	
MX2.1	Exposed to moisture but not exposed to freeze/thaw cycling or external sources of significant levels of sulphates or aggressive chemicals	Internal masonry exposed to high levels of water vapour, such as in a laundry. Masonry exterior walls sheltered by overhanging eaves or coping, not exposed to severe driving rain or frost. Masonry below frost zone in well-drained non-aggressive soil
MX2.2	Exposed to severe wetting but not exposed to freeze/thaw cycling or external sources of significant levels of sulphates or aggressive chemicals	Masonry not exposed to frost or aggressive chemicals, located: in exterior walls with cappings or flush eaves; in parapets; in freestanding walls; in the ground; under water
MX3	Exposed to wetting plus freeze/thaw cycling	
MX3.1	Exposed to moisture or wetting and freeze/thaw cycling but not exposed to external sources of significant levels of sulphates or aggressive chemicals	Masonry as class MX2.1 exposed to freeze/thaw cycling
MX3.2	Exposed to severe wetting and freeze/thaw cycling but not exposed to external sources of significant levels of sulphates or aggressive chemicals	Masonry as class MX2.2 exposed to freeze/thaw cycling
MX4	Exposed to saturated salt air, seawater or de-icing salts	Masonry in a coastal area. Masonry adjacent to roads that are salted during the winter
MX5	In an aggressive chemical environment	Masonry in contact with natural soils or filled ground or groundwater, where moisture and significant levels of sulphates are present
		Masonry in contact with highly acidic soils, contaminated ground or groundwater. Masonry near industrial areas where aggressive chemicals are airborne

Note: in deciding the exposure of masonry, the effect of applied finishes and protective claddings should be taken into account.

Clause 4.3.3(3)

Clause NA.2.11

If we next consider reinforced masonry, the units and the mortar will follow broadly the same approach as has been outlined above. Protection of the reinforcement, however, now needs to be considered. This is done in EN 1996-1-1, where the selection criteria for reinforcing steel for durability is given in a table in clause 4.3.3(3). The table in the code is a default table given in a Note, and national choice is permitted in determining the actual specification requirements. When using BS EN 1996-1-1 for design, the specification should be taken from clause NA.2.11 (Table 4.3) of the UK National Annex to EN 1996-1-1.

Figure 4.4. (a) Frost failure at the base of a free-standing wall, with moderately frost-resistant bricks. (b) General view of the wall, showing the castellated plan form

(a) (b)

Table NA.8

Table 4.3. Selection of reinforcing steel for durability

Exposure class[a]	Minimum level of protection for reinforcement, excluding cover	
	Located in bed joints or special clay masonry units	Located in grouted cavity or Quetta bond construction
MX1	Carbon steel galvanised in accordance with BS EN ISO 1461. Minimum mass of zinc coating 940 g/m^2 or for bed-joint reinforcement material/coating reference R1 or R3[b]	Carbon steel
MX2	Carbon steel galvanised in accordance with BS EN ISO 1461. Minimum mass of zinc coating 940 g/m^2 or for bed-joint reinforcement material/coating reference R1 or R3	Carbon steel or, where mortar is used to fill the voids, carbon steel galvanised in accordance with BS EN ISO 1461 to give a minimum mass of zinc coating of 940 g/m^2
MX3	Austenitic stainless steel in accordance with BS EN 10088 or carbon steel coated with at least 1 mm of stainless steel or for bed-joint reinforcement material/coating reference R1 or R3	Carbon steel galvanised in accordance with BS EN ISO 1461. Minimum mass of zinc coating 940 g/m^2
MX4 and MX5	Austenitic stainless steel[c] in accordance with BS EN 10088 or carbon steel coated with at least 1 mm of stainless steel or for bed-joint reinforcement material/coating reference R1 or R3[d]	Austenitic stainless steel[c] in accordance with BS EN 10088 or carbon steel coated with at least 1 mm of stainless steel[d]

[a] See EN 1996-2.
[b] In internal masonry other than the inner leaves of external cavity walls, carbon steel reinforcement or bed-joint reinforcement with any material/coating reference may be used.
[c] Austenitic stainless steel grades should be selected according to the exposure and environmental aggression applicable. Not all grades will necessarily be suitable for the most aggressive environments, particularly those environments where regular salts application is used as in highways de-icing situations.
[d] See BS EN 845-3.

Table NA.9

Table 4.4. Minimum concrete cover for carbon steel reinforcement

Exposure situations	Concrete grade in BS EN 206-1 and BS 8500				
	C25/30	C28/35	C32/40	C35/45	C40/50
	Minimum cement content[a]: kg/m^3				
	275	300	325	350	400
	Maximum free water/cement ratio				
	0.65	0.6	0.55	0.50	0.45
	Thickness of concrete cover: mm				
MX1[b]	20	20	20[c]	20[c]	20[c]
MX2	–	35	30	25	20
MX3	–	–	40	30	25
MX4 and MX5	–	–	–	60[d]	50

Data from NA to BS EN 1996-1-1:2005.
[a] With the exception of a 1:0 to ¼:3:2 cement:lime:sand:10 mm nominal maximum size aggregate mix, all mixes are based on the use of normal-weight aggregate of 20 mm nominal maximum size. Where other smaller-sized aggregates are used, cement contents should be adjusted in accordance with Table NA.10.
[b] Alternatively, 1:0 to ¼:3:2 cement:lime:sand:10 mm nominal aggregate mix may be used to meet exposure situation MX1, when the cover to reinforcement is 15 mm minimum.
[c] These covers may be reduced to 15 mm minimum provided that the nominal maximum size of aggregate does not exceed 10 mm.
[d] Where the concrete infill may be subjected to freezing while wet, air entrainment should be used.

Clause 4.3.3(4)
Clause NA.2.12

The values for minimum concrete cover c_{nom} for carbon steel reinforcement are given in clause 4.3.3(4) as a default table given in a Note, and national choice is permitted in determining the actual value for concrete cover. When using BS EN 996-1-1, the specification should be taken from clause NA.2.12 (Table 4.4) of the UK National Annex to EN 1996-1-1.

Table 4.4 should be used in conjunction with Table 4.5, which allows adjustments for variation in aggregate size.

It was the view of the committee that adopting the guidance given in the National Annex to BS EN 1996-1-1 should give the designer a broadly similar durability approach to that given in BS 5628.

Table NA.10

Table 4.5. Adjustments to minimum cement contents for aggregates other than 20 mm nominal maximum size

Nominal maximum aggregate size: mm	Adjustments to minimum cement contents in Table NA.9: kg/m^3
10	+40
14	+20
20	0

Data from NA to BS EN 1996-1-1:2005.

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Chapter 5
Structural analysis

5.1. General

Clause 5.1

This section requires little commentary: it lays down the principles and application rules by which the principles can be fulfilled. It is in major part in code language. *Clause 5.1(1)P*

Appropriate stability and robustness during construction as well as *during use* is required. *Clause 5.1(2)P*

The designer's attention is drawn to the fact that one person needs to have overall responsibility for the stability of the structure as a whole to ensure that, for example, if the walls are adequately designed by one designer and, say, the specialist space-frame roof by another, someone is responsible for ensuring that the roof is adequately fixed to the walls in a way that allows the whole building to work structurally. *Clause 5.1(3)*

The structure can be designed on linear elastic or non-linear plastic theory. If using the former, the short-term secant modulus of elasticity can be assumed. (See clause 3.7.2: $E = K_E f_k$, where K_E is an NDP. The recommended value in EN 1996 for K_E is 1000. This is also the value given to K_E in the UK National Annex (clause NA.2.9). BS 5628 used $E = 900f_k$, i.e. $K_E = 900$.) *Clause 5.1(4)* *Clause 3.7.2*

Clauses 5.1(5) and 5.1(6) lay out the important forces to consider in design and the fact that both ultimate and serviceability checks are required. *Clause 5.1(5)* *Clause 5.1(6)*

Having set down the principles and the ground rules, the code refers the designer to Sections 6 and 7 where design rules can be found that can be used to ensure that the principles are met. *Clause 5.1(7)*

5.2. Structural behaviour in accidental situations (other than earthquakes and fire)

Clause 5.2

Again, little commentary is required for this clause. *Clause 5.2(1)P* *Clause 5.2(2)*

Because earthquakes and fire have their own guidance documents, they are excluded from this section (but not, of course, from the designer's palette of tools).

The principle is first established that damage should not be disproportionate to the cause. This is the general *golden rule* for accidental damage design. *Clause 5.2(1)P*

Four methods are then offered by which structural members under accidental situations can be assessed/considered. These include (not in the same order as given in Eurocode 6):

- Isolating the structure from any source of likely potential damage: for example, vehicular impact barriers or bollards, or (not mentioned in the code) the use of a robust reinforced concrete or steel-framed structure for the ground floor where vehicular impact may be considered likely/possible.
- The use of alternative load paths by considering the notional removal of single load-bearing members one at a time in turn.
- Designing members to withstand accidental forces (see EN 1991-1-7). This is always a difficult route because of the lack of clarity concerning the magnitude of the forces to be used in design and the fact that these forces, when suggested, do tend to be large.

- The use of a tieing system to give extra strength and cohesive robustness to the structure. (This is also a delicate area, where too much tieing may be counter-productive and might be thought of as endangering the structure as a whole.)

The essence of the Eurocode 6 approach is to call on the designer to provide a masonry structure that is not only structurally stable but is also '*robust*' – that quality which is so difficult to define and even more difficult to quantify, but which can prove so useful when a structure is 'overtaken by events'.

Clause 5.3
Clause 5.3(1)
Clause 5.3(2)

5.3. Imperfections

The reader may be forgiven for wondering why imperfections should be considered with masonry when the global safety factor ($\gamma_f \gamma_m$) is normally of the order 4–6.

It appears that the arguments are somewhat different to these. The structure is assumed to be built imperfectly at an inclination (in radians) to the vertical given by

$$\nu = \frac{1}{100\sqrt{h_{tot}}} \tag{5.1}$$

where h_{tot} is the total height of the structure (in metres). Such an inclination gives horizontal forces, generated from the vertical forces present.

BS 5628 has always suggested that for normal design where shear walls provide the stability (i.e. sharing the total wind design moment on the building among the shear walls in proportion to their (structural) stiffness), the moment should be derived from the greater of:

- the design wind load or
- a horizontal force equal to 1.5% of the dead weight of the structure above the level being considered.

Eurocode 6 now requires only one check at any particular level – namely that the shear walls can sustain the moments generated by the *combined* horizontal loads due to wind and the horizontal component of the total dead load of the structure due to the assumed inclination.

Clause 5.4
Clause 5.4(1)
Clause 5.4(2)

5.4. Second-order effects

Most designers feel more confident when the design involves braced (non-sway) structures, which is the tradition on which masonry construction has developed in the UK. Clause 5.4 sets the principle that, if potentially present, sway must be taken account of in the design. A formula is offered to check that sway is not an influencing factor: the formula supplied for those who feel the need to perform this check.

No allowance for sway is necessary if the stiffening effect given by

Eqn 5.1

$$h_{tot} = \sqrt{\frac{N_{Ed}}{\sum EI}} \begin{cases} \leq 0.2 + 0.1n & \text{for } 1 \leq n \leq 4 \quad (5.2a) \\ \leq 0.6 & \text{for } n \geq 4 \quad (5.2b) \end{cases}$$

where:

n	is the number of storeys
h_{tot}	is the total height of the structure from the top of the foundation
N_{Ed}	is the design value of the vertical load (at the bottom of the building)
$\sum EI$	is the sum of the bending stiffnesses of all vertical stiffening building elements in the relevant direction

is satisfied.

Openings in vertical stiffening elements of less than 2 m^2 that have opening heights not exceeding $0.6h$ may be neglected.

Clause 5.4(2)

When the above condition is not satisfied, calculations should be carried out to check that any sway can be resisted. Following normal UK practice of avoiding sway masonry structures unless that they are, say, post-tensioned or reinforced, $\sum EI$ would require to be increased – perhaps by adjusting the wall thickness of some (or all) of the stiffening walls.

5.5. Analysis of structural members
Clause 5.5

5.5.1 Masonry walls subjected to vertical loading
Clause 5.5.1

5.5.1.1 General
Clause 5.5.1.1

The walls should be designed for the combination of the loads applied to them, any second-order effects, the eccentricities of the applied loads and any additional eccentricities that may be present (e.g. constructional tolerances). *Clause 5.5.1.1(1)*

The design of the wall is based on bending moments at the top, bottom and mid-height of the wall. Bending moments require to be calculated using structural mechanics applied to the relevant material properties and the joint behaviour. A method of doing this is given in Annex C. *Clause 5.5.1.1(2)*

The principle is set that in all wall designs an additional eccentricity, e_{init}, shall be assumed, to allow for construction imperfections. This should be applied throughout the full height of the wall. This is the only principle clause in this section. *Clause 5.5.1.1(3)P*

The initial eccentricity, e_{init}, may be assumed to be $h_{\mathrm{ef}}/450$, where h_{ef} is the effective height of the wall.

Commentary

EN 1996-1-1 uses a basic approach similar to that used in BS 5628 to calculate the vertical load resistance of the wall.

A reduction factor is applied to the strength that the wall would have if it were 'stocky' (i.e. no propensity to buckling-mode failure) and were perfectly axially loaded (i.e. no eccentricity of loading effect). The basic strength of the wall is based on the loaded cross-sectional area and its material strength. The capacity reduction factor recognises that the basic strength of the wall will be reduced by both:

- the slenderness of the member (normally masonry walls will not be 'stocky')
- the eccentricity of the applied load (normally the eccentricity will never be zero).

Thus, the design vertical load resistance is found using Φ, the capacity reduction factor.

$$\text{design vertical load resistance} = N_{\mathrm{Rd}} = \Phi A f_{\mathrm{d}} \tag{5.3}$$

Φ is the capacity reduction factor
A is the plan area of the wall being designed
f_{d} is the design compressive strength of the masonry.

(In BS 5628 the capacity reduction factor Φ was referred to as β.) *Clause 6.1.2.1(3)*

The determination of f_{d} has already been fully covered in Chapter 3. There is, however, an additional small-plan-area reduction factor that reduces the value of f_{d} slightly when the area of the wall/column is small:

$$\text{Design strength of masonry reduced for small plan area} = (0.7 + 0.3A)f_{\mathrm{d}} \tag{5.4}$$

Eqn 6.3

For unreinforced masonry, one of the areas covered in Chapters 5 and 6 is the method of determining Φ. To determine the value of Φ, the slenderness ratio and the eccentricity of applied load need to be determined. To find the slenderness ratio, both the effective height and the effective thickness of the wall must be found.

Clause 5.5.1.2

5.5.1.2 Effective height of masonry walls

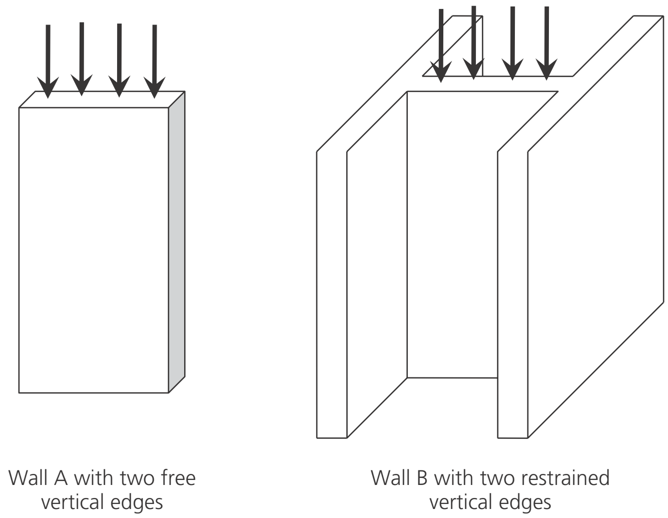
Commentary

Most engineers understand that effective height can deal with the different buckling strengths of struts (whether masonry or not) with different end conditions (i.e. different degrees of 'end fixity'). This principle is well established, and has been used in BS 5628.

Clause 5.5.1.2 (10)

EN 1996-1-1 also adopts this approach for vertical spanning members. In addition, however, it alters the effective height, depending on the degree of support of the two vertical edges of the wall. In a way, this may seem odd to many engineers since any additional load-carrying capacity that wall B in Figure 5.1 has over wall A will be considered by most engineers to be due to its geometry (i.e. to its 'structure and form'). Wall B will have a much enhanced L_{ef}/r: its structural slenderness will have changed and not just its effective height.

Figure 5.1. Effective heights of stiffened walls



The code chooses not to address walls this way (i.e. not to use an L_{ef}/r approach), and instead maintains the *slenderness ratio* approach adopted in BS 5628 while recognising, however, that supporting one or both of the vertical edges of the wall will beneficially alter the effective height: it permits this additional benefit to be used.

The principle is first set that the effective height should be assessed, 'taking account of the relative stiffness of the elements of structure connected to the wall and the efficiency of the connections'.

Clauses 5.5.1.2(1)P

Clauses 5.5.1.2(2)–5.5.1.2(9) then give guidance on minimum requirements (Figure 5.2). The guidance is both detailed and explicit, and requires little further elaboration here. The various clauses are summarised below:

Clause 5.5.1.2(2) gives the types of member that can stiffen a wall.

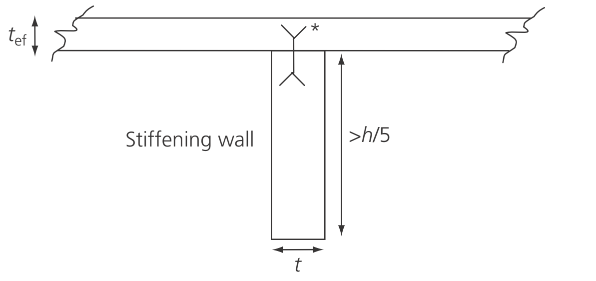
Clause 5.5.1.2(3) gives both limitations and methods to secure vertical edge restraint. The use of ties/anchors to support the vertical edge of a wall where a shrinkage crack may be anticipated in blockwork or calcium silicate brickwork construction is a practical way to sensibly recognise construction realities.

Clause 5.5.1.2(4) gives a minimum outstand length and thickness requirement for the stiffening wall.

Clause 5.5.1.2(5)

Clause 5.5.1.2(5) gives a relaxation of the above clause where there are windows or door openings in the stiffening wall. The limitations of the door or window openings are given. See Figure 5.1 in EN 1996-1-1.

Figure 5.2. Minimum requirements for assuming support to vertical edges of load-bearing walls



*Junction to be adequately tied if cracking is considered likely/possible

h is the clear height of the stiffened wall

t $\geq 0.3t_{ef}$

Clause 5.5.1.2(6) gives the designer the opportunity to use members other than masonry members to 'stiffen' a wall. It lays down the criterion that 'equivalent stiffness' should be used as the yardstick to judge whether a member is appropriate to stiffen the vertical edge/s of a wall. *Clause 5.5.1.2(6)*

Clauses 5.5.1.2(7) gives the length limitation whereby the middle of wall does not 'feel the presence of restraint' to its vertical edges. *Clause 5.5.1.2(7)*

The vertical restraint is not considered effective when:

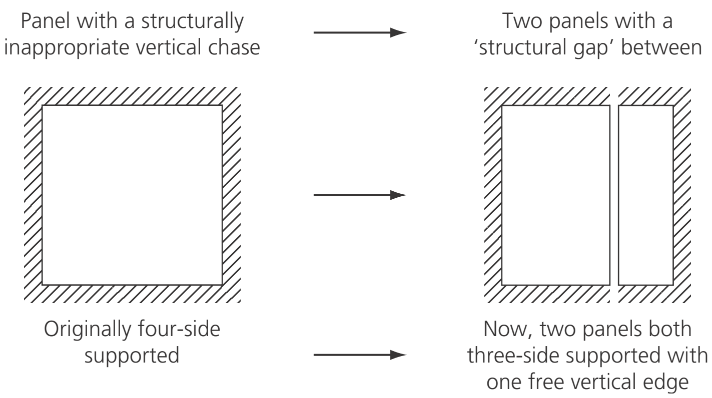
- $l \geq 30t$ – for walls stiffened on two vertical edges
- $l \geq 15t$ – for walls stiffened on one vertical edges.

Walls that fall outside these limits can thus only be supported on the top and bottom edges: the central portion does not 'feel' the effect of the stiffened vertical edge(s). *Eqn 5.2*

Clause 5.5.1.2(8) gives limitations whereby structurally inappropriate vertical chases should be seen as just that: they are considered to have introduced a sufficiently large weakness to effectively form a free vertical edge into the original panel, thus producing two smaller panels (either side) both with one free vertical edge (Figure 5.3). A chase is automatically assumed inappropriate if more than half of the wall thickness is removed by the chase. *Clause 5.5.1.2(8)*

Clause 5.5.1.2(9) follows the same argument as clause 5.5.1.2(8) above, but for openings. Where the openings are considered structurally inappropriate, a free edge should be assumed 'at the edge of the opening'. This clause gives the limitations on openings. The designer may care to explore a similar approach to that shown in Figure 5.3. *Clause 5.5.1.2(9)*

Figure 5.3. Dealing with an inappropriate vertical chase



Clause 5.5.1.2(10)

Following the general guidance and the limitation clauses above, guidance then moves to the determination of the effective height in clause 5.5.1.2(10).

This is dealt with in various equations in the code under clause 5.5.1.2(10). It is perhaps easier to explain the guidance as below:

- *Two-sided, vertically spanning walls* with both vertical edges free:

Eqn 5.2

$h_{ef} = \rho_2 h$ where ρ_2 is either 0.75 or 1, as given below (5.5)

Eqn 5.3

$\rho_2 = 0.75$ for walls restrained at the top and bottom by reinforced concrete floors or roofs spanning from both sides at the same level or by a reinforced concrete floor spanning from one side only and having a bearing of at least two-thirds of the thickness of the wall *and* for walls with a load eccentricity at the top that is not greater than $0.25t$

Eqn 5.4

$\rho_2 = 1.0$ for all other cases and for the case of reinforced concrete floors when the eccentricity $>0.25t$.

These are similar effective height factors to those given in BS 5628. They were based on the arguments below:

– assess whether any rotation (however small) might take place at the head of the wall
– if no rotation, then assume 'full fixity' at the wall ends, and use $\rho_2 = 0.75$
– if some rotation is present/possible (however small), then assume the 'pinned condition' and use $\rho_2 = 1.0$.

- *Three-sided, vertically spanning walls* with one vertical edge free and one restrained:

$h_{ef} = \rho_3 h$ (5.6)

where ρ_3 is given by

Eqn 5.6

$$\rho_3 = \frac{1}{1 + (\rho_2 h/3l)^2}\rho_2 \quad \text{where } h \leq 3.5l \text{ and } l < 15t \qquad (5.6a)$$

Eqn 5.7

$$\rho_3 = \frac{1.5l}{5} \geq 0.3 \quad \text{where } h \leq 3.5l \text{ and } l < 15t \qquad (5.6b)$$

ρ_3 can vary between 0.3 and 1.0.

- *Four-sided, vertically spanning walls* with both vertical edges restrained:

$h_{ef} = \rho_4 h$ (5.7)

where ρ_4 is given by the following. For vertically spanning walls with both vertical edges restrained:

Eqn 5.8

$$\rho_3 = \frac{1}{1 + (\rho_2 h/3l)^2}\rho_2 \quad \text{where } h \leq 1.15l \text{ and } l < 30t \qquad (5.7a)$$

Eqn 5.9

$$\rho_3 = \frac{0.5l}{5} \quad \text{where } h > 1.15l \text{ and } l < 30t \qquad (5.7b)$$

ρ_4 can vary between 0.1 and 1.0.

ρ_3 and ρ_4 are shown graphically in Annex D of the code.

Clause 5.5.1.3

5.5.1.3 Effective thickness of masonry walls

There is little surprise in this section of the Eurocode for designers who are used to BS 5628.

Clause 5.5.1.3(1)

A solid wall has an effective thickness of t, the actual thickness, if it is a single-leaf wall, a double-leaf wall, a faced wall, a shell-bedded wall or a grouted cavity wall (cavity width to be taken as 100 mm if it is greater than 100 mm (clause 5.5.2.1(2)); all are properly defined in clause 1.5.10.

Clause 5.5.1.3(2)

A wall stiffened by piers has an effective thickness enhanced by a factor given in Table 5.1 of EN 1996-1-1. The enhancement factor called ρ_t has a value greater than 1 but not more than 2:

Table 5.1. Limiting ratios of the effective span to the effective depth for walls subjected to out-of-plane bending and beams

Table 5.2

	Ratio of the effective span to the effective depth (l_{ef}/d) Or the effective thickness (l_{ef}/t_{ef})	
	Wall subjected to out-of-plane bending	Beam
Simply supported	35	20
Continuous	45	26
Spanning in two directions	45	–
Cantilever	18	7

Data taken from BS EN 1996-1-1:2005.

the exact value can be seen to depend on pier depth and pier spacing. This is a similar approach to that used in BS 5628.

A cavity wall has an effective thickness (similar to the $2/3(t_1 + t_2)$ rule from BS 5628) of

Clause 5.5.1.3(3)

$$t_{ef} = \sqrt[3]{k_{tef} t_1^3 + t_2^3} \tag{5.8}$$

Eqn 5.11

k_{tef} is to allow for the different E values of leaf t_1 and leaf t_2 (if any difference is present), and is defined as E_1/E_2

k_{tef} is an NDP; it has no recommended value (other than E_1/E_2) but has a recommended upper limit $= 2$.

The above formula is quite different to the $t_{ef} = 2/3(t_1 + t_2)$ rule given in BS 5628, but the result that the new formula gives is quite similar – as the comparison in Figure 5.4 shows.

In calculating the effective thickness of a cavity wall, it should be noticed that where one leaf is stiffened by piers the effective thickness of that leaf can be input as the thickness t_1 or t_2 – whichever applies. If, however, the unloaded leaf then becomes thicker than the loaded leaf, its thickness should be limited to the thickness of the loaded leaf when calculating the 'overall' effective thickness.

When only one leaf of a cavity wall is loaded, care is needed to ensure that the unloaded leaf does not adversely affect the loaded leaf. This refers to the difficulty that can arise when a cavity wall is constructed using two leafs of dissimilar material, such as expansive clay units and shrinking concrete or calcium silicate units. If flexible ties are used across the cavity, no problems should normally be anticipated. If solid-section inflexible ties are used, however, difficulties can arise because of the contra-movement of the two leafs and the inability of the tie to accommodate it.

Clause 5.5.1.3(4)

Figure 5.4. Comparison of t_{ef} derived from the Eurocode 6 and BS 5628 formulae

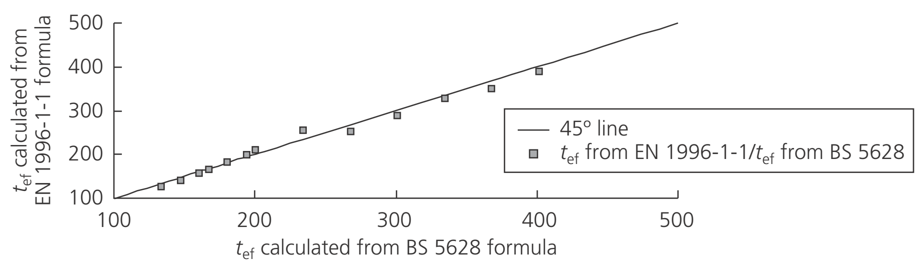
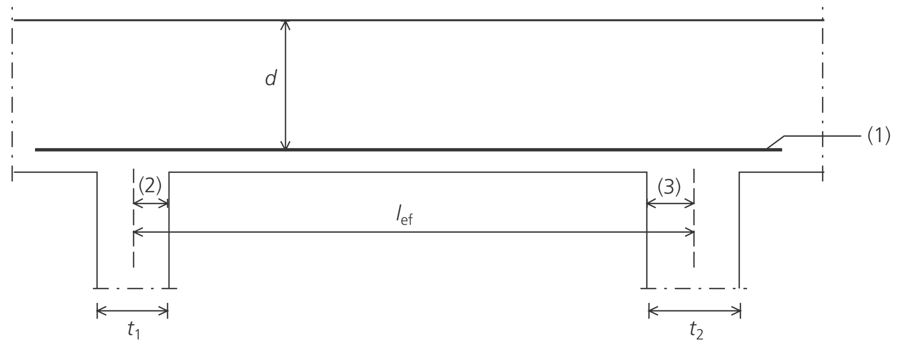


Figure 5.3

Figure 5.5. Effective span of simply supported or continuous masonry beams. (Reproduced from BS EN 1996-1-1 © British Standards Institution 2005)



Key
(1) Reinforcement
(2) $t_1/2$ or $d/2$ (whichever is the smaller)
(3) $t_2/2$ or $d/2$ (whichever is the smaller)

Clause 5.5.1.4

5.5.1.4 Slenderness ratio of masonry walls

When h_{ef} and t_{ef} have been determined, the slenderness ratio h_{ef}/t_{ef} can be established. A maximum limit of 27 is given:

$$\text{slenderness ratio} = \frac{h_{ef}}{t_{ef}} \quad (\leq 27) \tag{5.9}$$

This is the same maximum value used in BS 5628.

Clause 5.5.2
Clause 5.5.2.1
Clause 5.5.2.1(1)
Clause 5.5.2.1(3)

5.5.2 Reinforced masonry members subject to vertical loading

5.5.2.1 Slenderness ratio

The slenderness ratio of a vertically loaded reinforced masonry member should be determined using the same rules as previously given and discussed above (i.e. clause 5.5.1.4) with the maximum limit of slenderness ratio being 27 – the same as above.

Clause 5.5.2.1(2)

The effective thickness of grouted cavity wall construction is the actual thickness based on the width of the cavity being limited to 100 mm if the width is in excess of this value.

Clause 5.5.2.2

5.5.2.2 Effective span of masonry beams

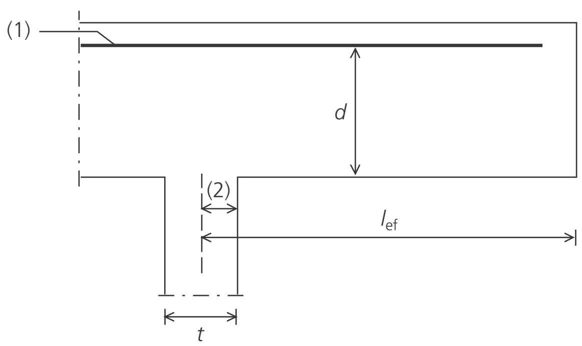
For non-deep beams that are simply supported or continuous (Figure 5.5), l_{ef} can be taken as the lesser of:

- distance between centre of supports or
- the clear distance between supports plus the effective depth, d.

For non-deep beams that form a cantilever (Figure 5.6), l_{ef} can be taken as the lesser of:

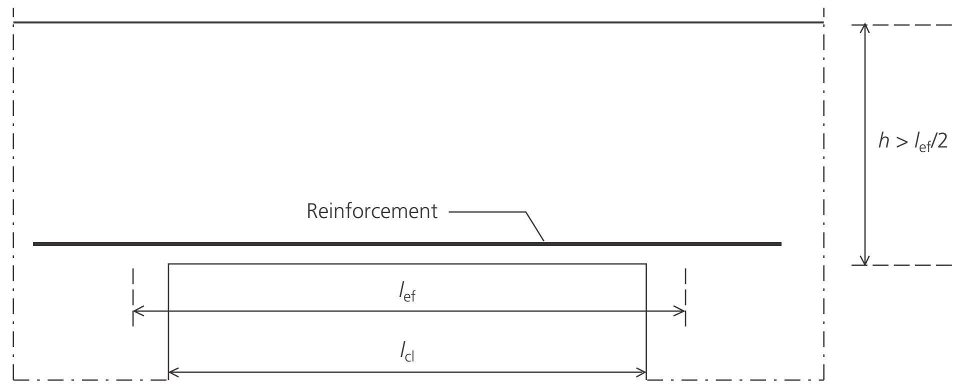
Figure 5.4

Figure 5.6. Effective span of masonry cantilever. (Reproduced from BS EN 1996-1-1 © British Standards Institution 2005)



Key
(1) Reinforcement
(2) $t/2$ or $d/2$ (whichever is the smaller)

Figure 5.7. Analysis of a deep masonry beam. (Reproduced from BS EN 1996-1-1 © British Standards Institution 2005) *Figure 5.5*



- distance between the centre of support and the end of the beam or
- the clear span of the cantilever plus one-half of the effective depth, d.

This is as described in BS 5628-2.

5.5.2.3 Deep masonry beams subjected to vertical loading

Clause 5.5.2.3

The effective span of a deep beam (Figure 5.7) may be taken as *Clause 5.5.2.3(1)*

$$l_{ef} = 1.15 l_{cl}$$

where l_{cl} is the clear width of the opening assuming that, by definition, a deep beam has a height given by *Clause 5.5.2.3(2)*

$$h > l_{ef}/2$$

In determining the bending moment in a deep beam, it can be assumed that it is simply supported. *Clause 5.5.2.3(3)*

5.5.2.4 Redistribution of internal forces

Clause 5.5.2.4

EN 1996-1-1 allows similar redistribution of internal forces as EN 1992-1-1.

5.5.2.5 Limiting spans of reinforced masonry members subjected to bending

Clause 5.5.2.5

Table 5.1 (Table 5.2 of EN 1996-1-1) gives limiting values for the effective span to effective depth or effective thickness ratios. These are guideline figures that are similar to those contained in BS 5628-2. They can be increased by 30% for isolated free-standing walls (i.e. detached from any part of a building structure). The note suggests that this enhanced figure should only be used if there is no applied finish to the wall that can be damaged by deflection. This would normally be the case in the UK, where most free-standing walls are unrendered.

In simply supported or continuous beams, the clear distance between lateral restraints, l_r, should be limited to the lesser of *Clause 5.5.2.5(2)*

$$l_r \leq 60 b_c \qquad (5.10)$$ *Eqn 5.13*

$$l_r \leq \frac{250}{d} b_c^2 \qquad (5.11)$$ *Eqn 5.14*

where b_c is the width of the compression face midway between restraints.

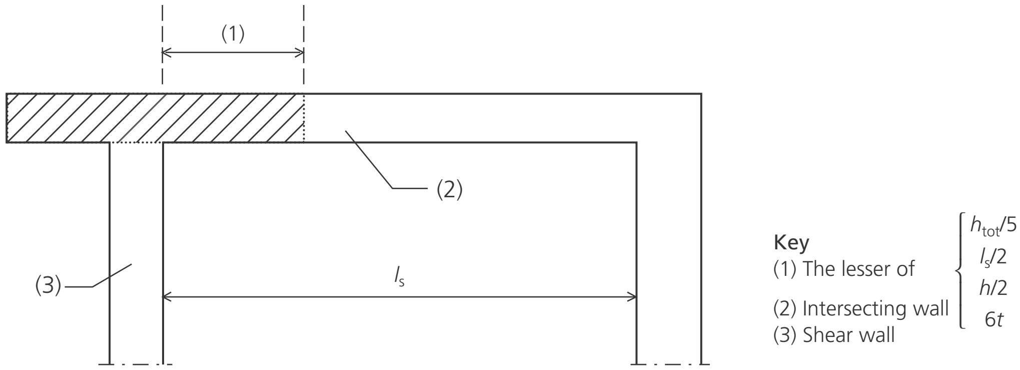
For cantilever beams where lateral support is only provided at the support, l, the clear distance from the end of the cantilever to the face of the support, l_r, is defined *Clause 5.5.2.3(3)*

$$l_r \leq 25 b_c \qquad (5.12)$$ *Eqn 5.15*

$$l_r \leq \frac{100}{d} b_c^2 \qquad (5.13)$$ *Eqn 5.16*

Figure 5.6

Figure 5.8. Flange widths that can be assumed for shear walls. (Reproduced from BS EN 1996-1-1 © British Standards Institution 2005)



Clause 5.5.3

Clause 5.5.3(1)

5.5.3 Masonry walls subjected to shear loading

When analysing masonry walls subjected to shear loading, the elastic stiffness of the walls, including any flanges, should be used as the stiffness of the wall. For walls higher than twice their length, the effect of shear deformations on the stiffness can be neglected.

Clause 5.5.3(2)

Provided that the connection is structurally adequate, a section can be considered as a shear wall together with any flange(s) from intersecting walls – as is the case with BS 5628. Of course, the flange must not buckle within the length assumed.

Clause 5.5.3(3)

Limits to the length of outstand that can be assumed for design purposes are given (Figure 5.8). This should be the least of:

- $h_{tot}/5$, where h_{tot} is the overall height of the shear wall
- half the distance between shear walls (l_s), when connected by the intersecting wall
- the distance to the end of the wall
- half the clear height (h)
- six times the thickness of the intersecting wall, t.

Clause 5.5.3(4)

A relaxation is given for the case of intersecting walls with openings where the dimensions are smaller than $h/4$ or $l/4$. It is suggested that these openings may be disregarded. Openings with dimensions greater than $h/4$ or $l/4$ should be regarded as marking the end of the wall. It is not clear to the writer where this relaxation stems from. A designer may wish to end the wall at the beginning of an opening, irrespective of size, as presumably would be the case using BS 5628.

Clause 5.5.3(5)

The horizontal forces may be distributed to the shear walls in proportion to their stiffness, but only if the floors can act as rigid diaphragms.

Clause 5.5.3(6)P

The principle is introduced that the effect of torsion shall be considered when the plan arrangement of the shear wall assemblages is asymmetric. Presumably, a matter of 'degree' comes into this since virtually no building plan-form provides *complete* symmetry.

Clause 5.5.3(7)

Clause 5.5.3(7) is self-explanatory and follows on from (5) above.

Clause 5.5.3(8)

Clause 5.5.3(8) is self-explanatory although, other than the *general approximation* of the structural engineering process, the writer is unaware of the reason for this clause.

Clause 5.5.3(9)

Clause 5.5.3(9) states what most designers would tend to do. The reference to a 45° spread of load is, however, useful.

Clause 5.5.3(10)

The fact that the distribution of shear stress along the compressed part of a wall may be assumed to be constant describes the current BS 5628 approach: the shear stress across the section is maintained as uniform.

5.5.4 Reinforced masonry walls subjected to shear loading

Clause 5.5.4

The clauses regarding shear at the end support (and at intermediate supports if any are present) need little comment.

5.5.5 Masonry walls subjected to lateral loading

Clause 5.5.5

Commentary

A reader who is familiar with the BS 5628 approach used for the design of laterally loaded panels will feel comfortable and familiar with the approach contained in Eurocode 6. It is essentially very similar with only the occasional difference – some of which make design easier.

Clause 5.5.5(1)

The designer's attention is first drawn to the possible effect of:

- Any damp-proof membrane/courses being present. These may reduce the panel edge condition to a simple support (membrane).
- Continuity over supports. (Here, the edge restraint may become fully fixed.)

Clause 5.5.5(2)

Clauses 5.5.5(2), 5.5.5(3) and 5.5.5(4) are self-explanatory.

Clause 5.5.5(3)

There is a note under clause 5.5.5(3) regarding the use of specialised anchors. Common examples of this in the UK are:

- in free-standing walls, connecting a leaf across a movement joint and into a pier providing support/stability to that end of the wall panel (here, sleeved (i.e. de-bonded) ties have been used for many years)
- when there is a horizontal movement joint at the top of a panel.

The writer is not aware why these products fall outside the scope of Eurocode 6: it appears that they have been widely used in the UK in a highly practical way and, to the writer's knowledge, they do not appear to have been problematic.

Clause 5.5.5(4)

Clause 5.5.5(4) is useful in making design simpler.

Clause 5.5.5(5)

Clause 5.5.5(5) elaborates on which edge conditions may be considered simple supports or fully fixed (i.e. continuous).

Clause 5.5.5(6)

Clause 5.5.5(6) permits continuous design of a cavity wall even though only one leaf is actually continuous. The more onerous condition in BS 5628 that the thicker leaf (if there is one) must be the continuous leaf has been removed. Furthermore, the panel can be supported by ties to only one leaf. This is most useful when considering masonry cladding to steel-framed buildings where ties between the outer masonry leaf and the structural steelwork are positively discouraged, since they provide a possible direct water path from the outer leaf to the steel.

Clause 5.5.5(7)

Clause 5.5.5(7) is the basis of the ultimate (and serviceability) design using the α bending moment coefficients, which are similar to those in BS 5628. While recognising much of the approach adopted by Eurocode 6, designers will see that it has been written in a style that offers them the ability to check either vertically (using α_1 and f_{xk1}) or horizontally (using α_2 and f_{xk2}).

Clause 5.5.5(8)

Clause 5.5.5(8) is self-explanatory: full fixity could be assumed across a brick damp-proof course.

Clause 5.5.5(9

Clause 5.5.5(9) follows on from the last clause in that, when referring specifically to a vertical one-way spanning wall, it may be designed as a simply supported beam or as a propped cantilever.

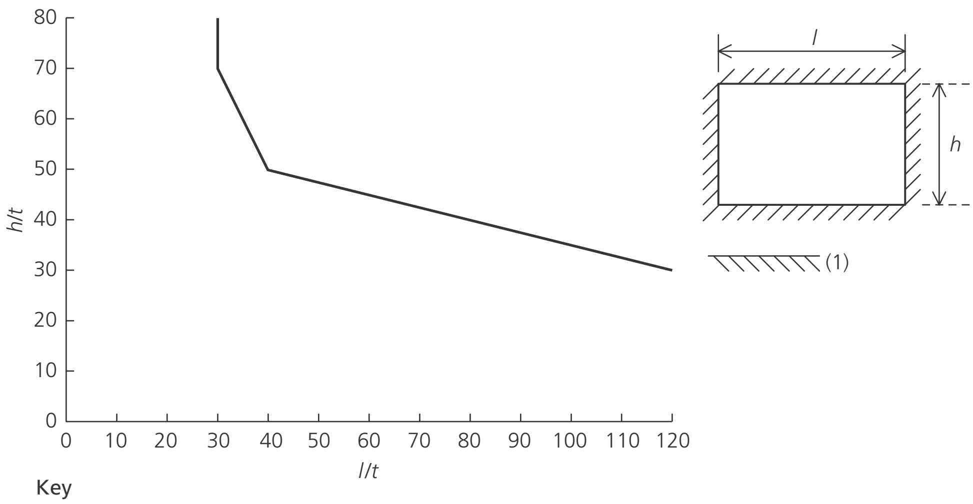
Clause 5.5.5(10)

Clause 5.5.5(10) introduces the concept of 'limit of size' on the panel dimensions (Figure 5.9). The limits to dimensions are given in Annex F in graphical form for:

- four-side-supported panels

Figure F.1

Figure 5.9. Limiting height and length to thickness ratios of walls restrained on all four edges. (Data taken from EN 1996-1-1)



Key
(1) Simply supported or with full continuity

- three-side-supported C-shaped panels
- three-side-supported U-shaped panels.

Annex F

Commentary

The limits given in Annex F of EN 1996-1-1 could be thought of as a form of 'serviceability limit state' although, strictly speaking, the load that causes the first crack in a laterally loaded panel is both the serviceability load *and* the ultimate load. A more correct way of looking at this would be to describe the limits of size as those limits that broadly describe the limits within which experimental evidence has been obtained. This usually relates to the size of panel we are used to meeting in practice. At the time of writing, there is an awareness within the UK masonry world that the size of some of the panels that can pass the Annex F test are really quite large – indeed, somewhat larger than we are used to. Bearing this in mind, judgement should be used if the designer has concern about certain large panels that appear to be satisfactory to Annex F requirements but cause the designer unease. A change currently being discussed that has much support among BSI mirror committee members is the concept of reintroducing the limit on the length and height of a panel equal to $50t_{ef}$. There may be changes to the National Annex that, in the future, could reduce the maximum size of panels derived solely from Annex F.

Clause 5.5.5(11)

Finally, in clause 5.5.5(11), for panels of irregular shape or with substantial openings Eurocode 6 permits the designer to use some other recognised tool of analysis such as finite-element or yield line analogy that can permit the anisotropic nature of masonry to be taken into account. Presumably, this will enable designers to use appropriate software and facilitate software houses to develop it or extend the BS 5628 packages already available to cover Eurocode 6.

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Chapter 6
Ultimate limit state

6.1. Unreinforced masonry walls subjected to mainly vertical loading
Clause 6.1

6.1.1 General
Clause 6.1.1

The principle is first established that the resistance of masonry walls to vertical loading shall be based on:

- geometry of the wall (i.e. its slenderness)
- the effect of applied eccentricities
- the material properties of the masonry (particularly f_k and E).

This of course is the engineering basis on which any strut in any material is designed. In developing this principle, no tension may be assumed and plain sections are assumed to remain plain.

6.1.2 Verification of unreinforced masonry walls subjected to mainly vertical loading
Clause 6.1.2

6.1.2.1 General
Clause 6.1.2.1

Clause 6.1.2.1(1)P

The other important principle is that the design load applied shall always be less than the design load resistance:

$$N_{Ed} \leq N_{Rd} \tag{6.1}$$

Eqn 6.1

Again, this is true for any strut of any material.

The moment assumed to be the wall is shown in Figure 6.1. Three critical positions require to be examined:

- the stress block at the top of the wall
- the stability condition at the mid height of the wall
- the stress block at the base of the wall.

As mentioned earlier, the general approach uses a capacity reduction factor:

$$N_{Rd} = \Phi t f_d \tag{6.2}$$

Eqn 6.2

Clause 6.1.2.1(2)

where

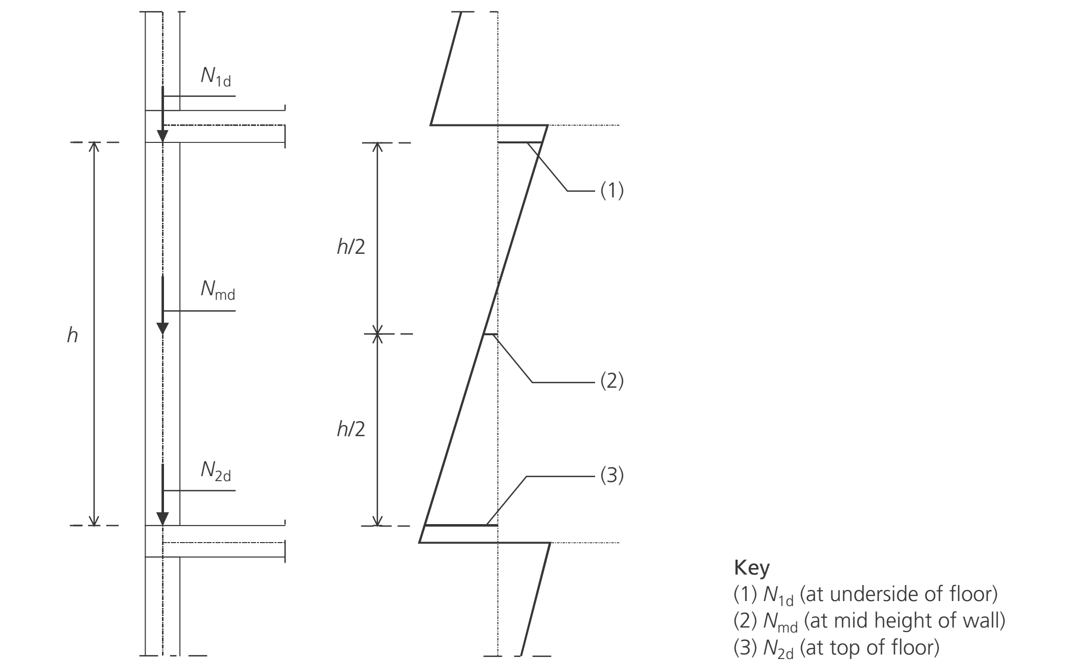
Φ is the governing capacity reduction factor (i.e. the smallest value of Φ) at the top, at the bottom or at the middle of the wall allowing for the combined effect of slenderness or eccentricity)
t is the thickness of the wall
f_d is the design compressive strength (f_k/γ_m).

Clause 6.1.2.1(3)

A small-area reduction factor should be applied where the cross-sectional area of a wall is less than 0.1 m^2 to recognise that, in struts of small cross-sectional areas, the presence of a single 'low-strength' structural unit has an unduly large effect when compared with such a unit being present within a wall with a large plan area. The small-area reduction factor lowers the design compressive strength of the masonry, f_d. This is similar to the approach used in BS 5628.

Figure 6.1

Figure 6.1. Moments from the calculation of eccentricities. (Reproduced from BS EN 1996-1-1 © British Standards Institution 2005)



Clause 6.1.2.1(4) The design of cavity walls should be based on an individual check of each loaded leaf for the area of the loaded leaf only, but assuming a slenderness ratio based on the effective thickness of the cavity wall.

Clause 6.1.2.1(5) When a faced wall is being designed, it should be designed by assuming that the whole single-leaf wall is constructed entirely of the weaker units. The appropriate K factor (from Table 3.3 in EN 1996-1-1 or Table NA.4 in its National Annex) should also be used (i.e. K should be taken for the weaker unit using both the material and grouping for that unit; because there will be a longitudinal mortar joint, K should also be multiplied by a factor of 0.8).

Clause 6.1.2.1(6) A double-leaf wall (previously referred to as a collar-jointed wall) can be designed either as a single-leaf wall or as a cavity wall, depending on whether both leaves have a load of similar magnitude. If designed as a solid wall, the comments immediately above on the K factor will also apply.

Clause 6.1.2.1(7) Where walls contain structurally inappropriate vertical chases (i.e. where walls are deeply chased or where the chases fall outside the recommendations given in clause 8.6) the effect can be taken into account by assuming that (Figure 6.2):

Figure 6.2. Dealing with an inappropriate vertical chase.

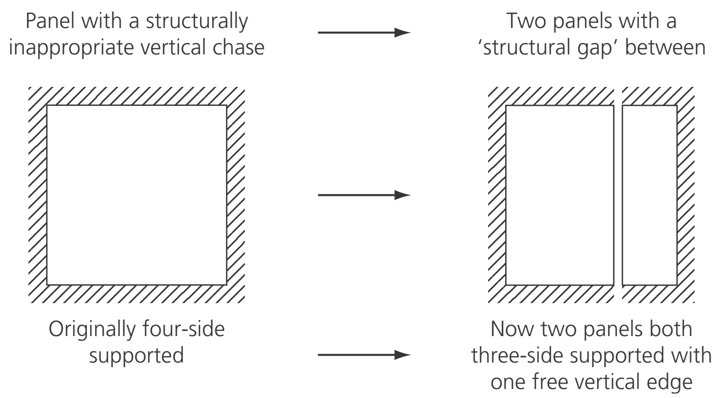
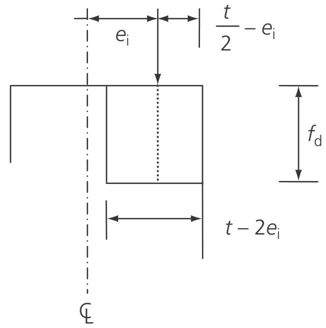


Figure 6.3. Stress block at the head of a wall as assumed for the derivation of Φ



- the wall effectively ends at the side of the vertical chase and, presumably, a new wall starts again at the other side of the chase or
- the wall has a real thickness equal to its actual thickness minus the depth of chase or
- the eccentricity is altered to account for the chase being present.

Great care is required with horizontal or inclined chases since the structural effect of horizontal or inclined chases is so much more profound than if the chases were vertical.

A Note usefully suggests that as long as the cross-sectional area of material 'removed' by the forming of a chase does not exceed 25% of the original wall, the reduction in load-bearing capacity may be reduced in proportion to the reduction in the cross-sectional area. Experienced designers may wish to check this approach by calculation until they become familiar with it.

6.1.2.2 Reduction factor for slenderness and eccentricity

Clause 6.1.2.2
Clause 6.1.2.2(1)(i)

The method of calculating the capacity reduction factor at the top of the wall is identical to the method used in BS 5628.

At the top or bottom of the wall (Figure 6.3), Φ_i is derived as follows:

$$N_{Rd} = (t - 2e_i)\, f_d$$
$$= \left(1 - 2\frac{e_i}{t}\right) t f_d$$
$$= \Phi_i t f_d$$

where

$$\Phi_i = 1 - 2\frac{e_i}{t} \qquad (6.3)$$

where

e_i is the structural eccentricity present at the top or bottom of the wall from floor and wall dead weights + additional structural eccentricities (e.g. from wind) + initial eccentricity ($e_{init} = h_{ef}/450$).

So,

$$e_i = \frac{M_{id}}{N_{id}} + e_{he} + e_{init} \geq 0.05t \qquad (6.4)$$

where

M_{id} is the design value of the bending moment at the top or the bottom of the wall resulting from the eccentricity of the floor load at the support, analysed according to clause 5.5.1 (Figure 6.1)

N_{id} is the design value of the vertical load at the top or bottom of the wall
e_{he} is the eccentricity at the top or bottom of the wall, if any, resulting from horizontal loads (e.g. wind)
e_{init} is the initial eccentricity (see clause 5.5.1.1)
t is the thickness of the wall.

Clause 6.1.2.2(1)(ii)

It is acknowledged that the load can never be applied perfectly axially, and a minimum eccentricity of 5% of the thickness ($0.05t$) has been adopted. This is an identical approach to the minimum eccentricity adopted in BS 5628.

At the middle of the wall height, Φ_m is derived using a more complex approach.

The theory that was adopted by the committee was developed by Kukulski and Lugez. It was developed and published in 1966, and was intended to address the design of complex-shaped reinforced concrete sections in compression. It therefore had to be modified quite considerably in order to fit not only to a masonry model but also to 'plate' design. The extent of this can be seen in Annex G of EN 1996-1-1 (which is an informative annex): the complexity of the equations makes the ready calculation of Φ_m difficult for desktop design, although it is very suitable for software. The first part of the annex deals with the general case of material of any E value. The second part of the annex offers the same formula specifically tailored for two values of E: one for $E = 1000f_k$ (the recommended default figure for E) and one for $E = 700f_k$. Of more practical interest for desktop design, values of Φ_m are presented graphically in Annex G for these two values of E.

In Annex 4 of this guide, tabular values for Φ_m are presented. The tables give values of Φ_m for masonry with E values between $E = 1000f_k$ and $E = 200f_k$. These are meant as an aid for designers because of the complexity of the calculation: they are also meant to be helpful in allowing designers to readily establish the scale of influence of the value of E on the compressive strength of a wall or pier when, for example, undertaking restoration work in old, 'soft' masonry.

The value of Φ_m requires the appropriate eccentricity at the wall mid-height e_{mk} to be first established. Calculation of e_{mk} includes contributions from:

- the structural eccentricity from floors and walls above
- additional structural eccentricities (e.g. from wind)
- initial eccentricities (e_{init})
- eccentricities due to creep (e_k).

e_{mk}, the eccentricity at the middle height of the wall, is calculated from the two equations below:

Eqn 6.6

$$e_{mk} = e_m + e_k \geq 0.05t \tag{6.5}$$

and

Eqn 6.7

$$e_m = \frac{M_{md}}{N_{md}} + e_{hm} \pm e_{init} \tag{6.6}$$

where

e_m is the eccentricity due to loads
M_{md} is the design value of the greatest moment at the middle of the height of the wall resulting from the moments at the top and bottom of the wall (see Figure 6.1), including any load applied eccentrically to the face of the wall (e.g. brackets)
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 (e. g. brackets)
e_{hm} is the eccentricity at mid-height resulting from horizontal loads (e.g. wind) (Note: the inclusion of e_{hm} depends on the load combination being used for the verification; its sign relative to that of M_{md}/N_{md} should be taken into account)
e_{init} is the initial eccentricity (see clause 5.5.1.1) ($e_{init} = h_{ef}/450$)

h_{ef} is the effective height, obtained from clause 5.5.1.2 or the appropriate restraint or stiffening condition
t_{ef} is the effective thickness of the wall, obtained from clause 5.5.1.3
e_k is the eccentricity due to creep, calculated from the Equation (6.8) of EN 1996-1-1,

$$e_k = 0.002\phi_\infty \frac{h_{ef}}{t_{ef}}\sqrt{te_m} \quad (6.7)$$

Eqn 6.8

ϕ is the final creep coefficient (see note under clause 3.7.4(2)).

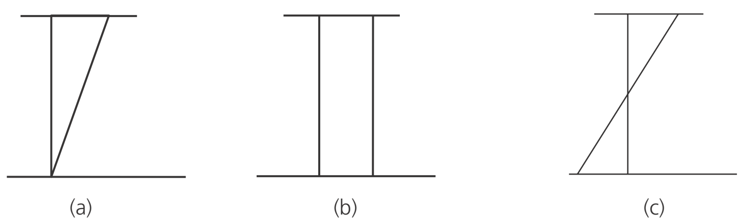
Clearly, the effect of creep becomes more pronounced with an increase in the load (and therefore stress) and an increase in the slenderness. This is recognised by suggesting that when the slenderness ratio is less than a critical number λ_c, the creep eccentricity may be taken as zero. The value of λ_c is an NDP: its value will be found in the National Annex. The recommended default value of λ_c is 15. The UK National Annex has a value of 27 for the slenderness ratio, λ_c, beneath which creep eccentricities do not need to be taken into account. e_k can therefore be taken as zero in UK.

Clause NA.2.14

Commentary
Calculation of the structural eccentricity
In order to use the above approach, the structural eccentricity requires to be found.

In a sense, this is not really a matter for codes but for the engineer to assess from their structural knowledge and training. Examples are shown in Figure 6.4. All of these structural models can apply to masonry. Figure 6.4(a), for example, could relate to a simple detached garage wall where nothing but the base of the wall connects to the footing and where no rotation of the footing should therefore take place since there is no force available to rotate it.

Figure 6.4. Eccentricity diagrams for different forms of masonry constructions. (a) A base-pinned stanchion. (b) A post-tensioned diaphragm wall, describing the condition at the transfer of post-tensioning stress. (c) A strut where the top of the strut is rotated by the same amount as at the bottom and in the same direction



If, on the other hand, a two-storey structure were built with in situ floors and the floor props removed, the floors would deflect causing imperceptible rotation at the end of the floors. This in turn would cause small wall deflections that would generate moments. These deflected shapes are shown, in a grossly exaggerated way, in Figure 6.5(a). So, wall 2 in the figure would become a double-curvature wall while wall 1 remains in single curvature (Figure 6.5(b)).

It is worth considering what transpires when precast units are used without propping. Figures 6.6(a) and 6.6(b) show the application of the precast units to the head of the lower wall (wall 1) before and after it is released by the crane (again, deflected shapes are grossly exaggerated for clarity). Wall 1 goes into single curvature. When the next floor is lifted onto wall 2, Figures 6.6(c) and 6.6(d) show the deflected shapes before and after it is released by the crane. All walls are now in single curvature due to the dead loads of the units.

Of course there will be imposed load on the floors under normal circumstances but generally with masonry construction the live load is subservient to the dead weight of the structure.

Figure 6.5. Wall eccentricities with in situ reinforced concrete floors

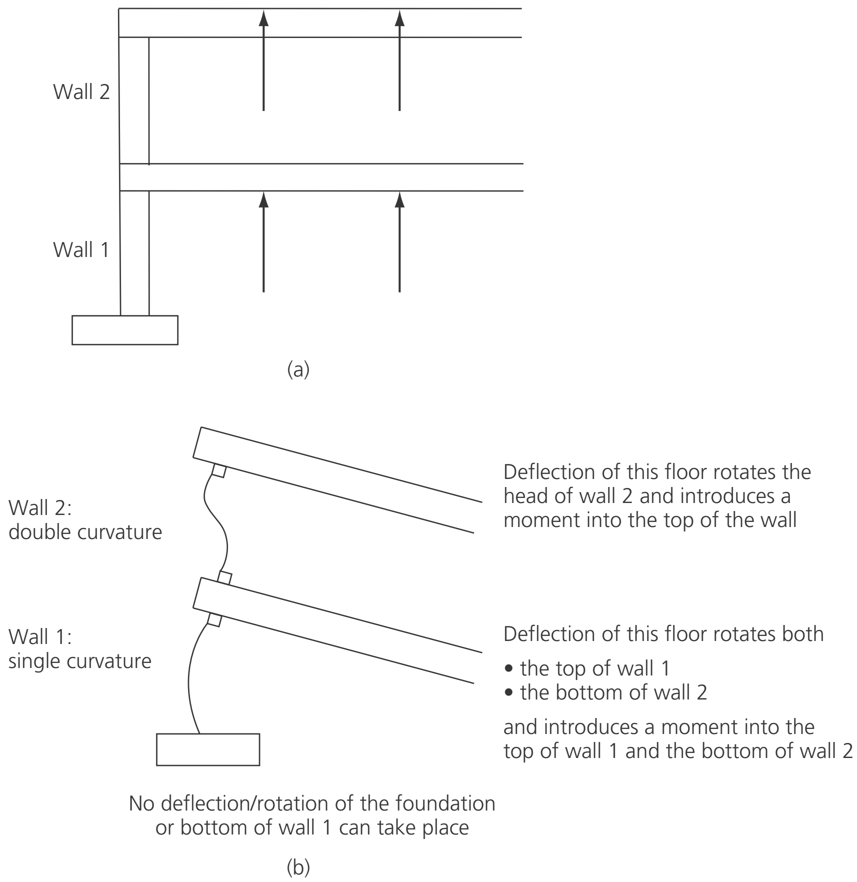
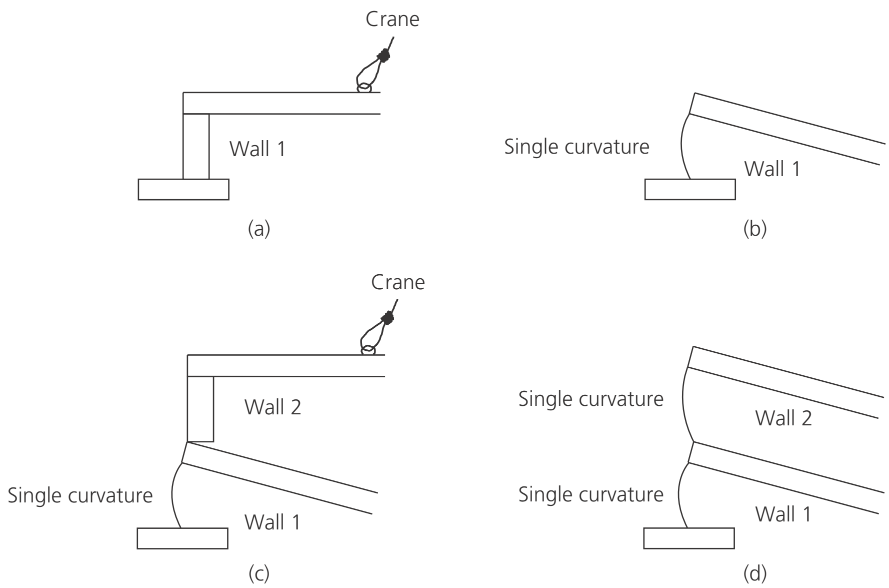
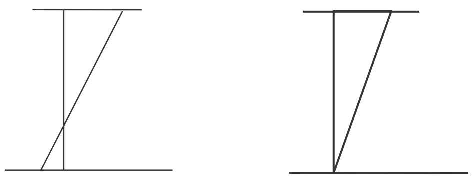


Figure 6.6. Wall eccentricities with precast concrete floors. (a) Precast unit on the wall but not yet fully released by the crane. (b) Precast unit on the wall and fully released by the crane. (c) Next floor precast unit on the wall but not yet fully released by the crane. (d) Both floors now fully released by the crane



Taking a small amount of imposed loading into account, the eccentricity diagram due to structural loads, will be similar to Figure 6.7(a). This becomes even more complex to alculate, and the approximation of using the eccentricity diagram (Figure 6.7(b)) is both easier and lower bound.

Figure 6.7. Actual and possible-assumed wall eccentricities with precast floors with dead and imposed loads



The designer should therefore be aware what type of construction is envisaged, since this will impinge on the form of the eccentricity diagram to be used. The reader will have noticed already that the moment (or eccentricity) at the head of the wall is not affected: it is the eccentricity *at the middle of the wall* that will be altered, depending on whether the designer believes the building will have walls subject to a single- or double-curvature model.

Where the walls are internal with floors either side and of roughly equal span, the eccentricity in the wall will tend to be small anyway. The above arguments will therefore have the greatest effect on the design of external walls and, depending on the details employed, virtually every 'ground floor' wall (i.e. those walls which are taken to the foundations that cannot readily be rotated by the ground floor slab detail).

With this background, we can now look at what the code proposes for the calculation of eccentricity.

The only principle clauses that impinge on walls vertically loaded by floors state: *Clause 5.5.1.1(3)P* *Clause 6.1.2.1(1)P*

- the use of e_{init}
- that $N_{\text{Ed}} \leq \text{N}_{\text{Rd}}$.

The code goes on to offer the designer a double-curvature model. It is not a principle, however: the designer is still able to choose the model which best describes his design (see Figure 6.1).

Let us assume, for the moment, that double curvature is the correct model for the wall being considered. Annex C (an informative annex that may be used in the UK) offers a simplified method for calculating the out-of-plane eccentricity. *Clause C.2*

The moment at the head of the wall is first calculated using moment distribution:

$$M_1 = \frac{\dfrac{n_1 E_1 I_1}{h_1}}{\dfrac{n_1 E_1 I_1}{h_1} + \dfrac{n_2 E_2 I_2}{h_2} + \dfrac{n_3 E_3 I_3}{h_3} + \dfrac{n_4 E_4 I_4}{h_4}} \left(\frac{w_3 l_3^2}{4(n_3 - 1)} - \frac{w_4 l_4^2}{4(n_4 - 1)} \right) \tag{6.8}$$

Eqn C.1

It is believed that the above equation will usually overestimate the moment and the eccentricity should be reduced by multiplying it by η, which may be taken as $(1 - k_{\text{m}}/4)$, where

$$k_{\text{m}} = \frac{n_3 \dfrac{E_3 I_3}{l_3} + n_4 \dfrac{E_4 I_4}{l_4}}{n_1 \dfrac{E_1 I_1}{h_1} + n_2 \dfrac{E_2 I_2}{h_2}} \leq 2 \tag{6.9}$$

Eqn C.2

Having found M at the head of the wall, M at the base of the wall next requires to be found in a basically similar approach by simply 'moving' the equation 'down the building' by one wall.

Once M at the top and bottom of the wall is found, the structural eccentricity M/N can be found for the top of the wall and for the bottom of the wall. This then requires to be added to the initial eccentricity e_{init} and any wind eccentricity e_{he} at the top or the bottom of the wall. *Clause 6.1.2.2(1)(ii)*

Clause 6.1.2.2(2)
Clause NA.2.1.4

M at the middle of the wall is then taken as the algebraic sum of the top and bottom moments. In double curvature, this will give a small M at the middle of the wall. (In single curvature, M at the middle of the wall is taken as half M at the top of the wall, and M at the base of the wall does not need to be established.)

Annex G(i)
Annex G(ii)

Once M at the middle of the wall is found, the structural eccentricity M/N can be found for the middle of the wall. This then requires to be added to the initial eccentricity e_{init} and any wind eccentricity e_{hm}. The creep eccentricity also requires to be added unless, as is the case in the UK, it is not required to be considered for the wall in question by virtue of the National Annex.

Φ_{m} can then be established by reading it off either Figure G.1 or Figure G.2 in Annex G.

A design example using this approach is given in Section A5.1 of Annex 5 of this guide. It is probable that this approach will be the default approach in software programmes for the design of walls in compression.

A note in Annex C of EN 1996-1-1 suggests that the above-moment distribution model is not appropriate for timber floor construction. The reader may also consider that the model is inappropriate for other forms of light floor or roof systems.

Clause 5.5.1.1(3)
Clause 5.5.1.1(1)

The concept of using the established UK practice of calculating eccentricities in the simple, traditional way of assuming forces to act at the third of the bearing in from the face of the wall may prove useful here. A design example using this method of calculating eccentricities is also given in Section A5.2 of this guide. Questions 1 and 2 use the double-curvature model while Question 3 uses single curvature. The designer will need to consider which is the appropriate model to use for the design details pertaining. This simple approach of assessing the structural eccentricity may prove the more popular method for desktop design because it is a shorter calculation and one with which British designers are accustomed. Assuming that the initial eccentricity e_{init} is also used, the one principle clause (clause 5.5.1.1(3)) is thereby met as are the other (non-principle) suggestions given in clause 5.5.1.1(1).

Clause 6.1.3

6.1.3 Walls subjected to concentrated loads

The principle is first established that the design value of the vertical concentrated load resistance of the wall shall always exceed the design value of the concentrated applied vertical load:

Eqn 6.9

$$N_{\text{Edc}} \leq N_{\text{Rdc}} \tag{6.10}$$

Commentary

It has often been suggested by designers that the rules governing concentrated loads in BS 5628 were somewhat confusing and lacking in clarity. This cannot be said of Eurocode 6, where further research has led both to greater understanding and to a more rigorous approach.

If we deal with the vertical concentrated load generally for the moment,

Eqn 6.10

$$N_{\text{Rdc}} = \beta A_{\text{b}} f_{\text{d}} \tag{6.11}$$

where

β is an enhancement factor for concentrated loads,

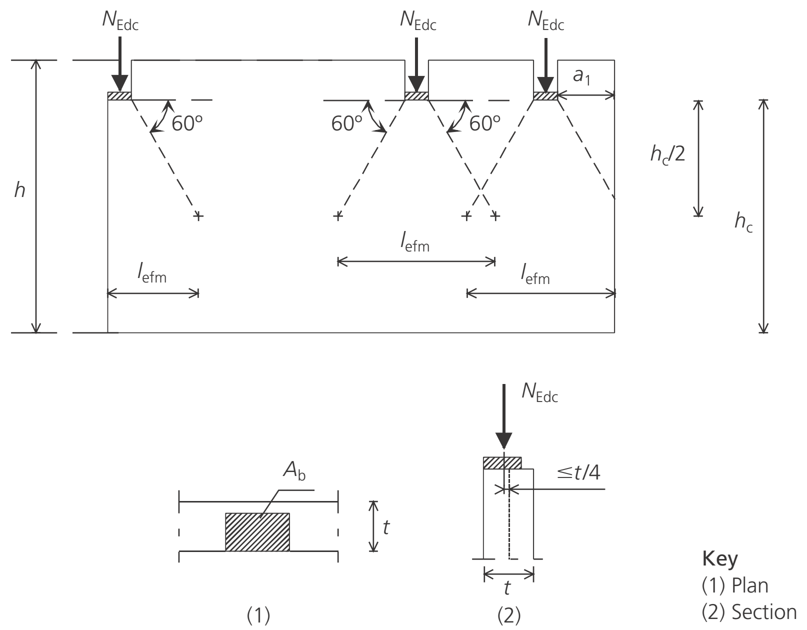
Eqn 6.11

$$\beta = \left(1 + 0.3\frac{a_1}{h_{\text{c}}}\right)\left(1.5 - 1.1\frac{A_{\text{b}}}{A_{\text{ef}}}\right) \quad \text{for group units} \tag{6.12}$$

and should not be less than 1.0 nor taken to be greater than

$$1.25 + \frac{a_1}{2h_c} \quad \text{or} \quad 1.5 \qquad \text{whichever is the lesser}$$

Figure 6.8. Walls subjected to concentrated load. (Reproduced from BS EN 1996-1-1 © British Standards Institution 2005) *Figure 6.2*

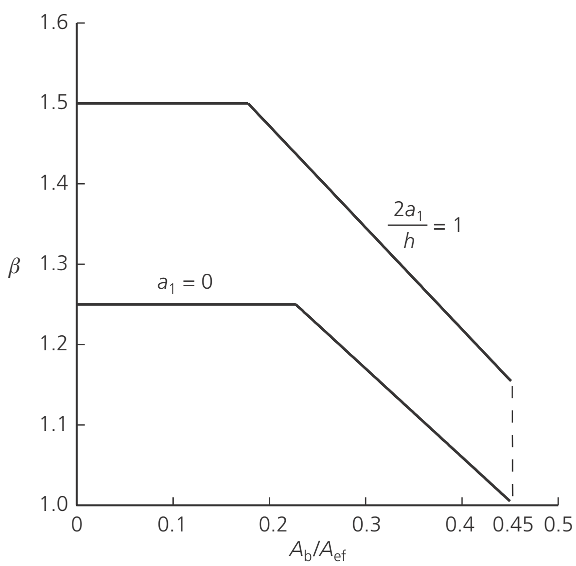


A_b is the loaded area
f_d is the design compressive strength of the masonry
a_1 is the distance from the end of the wall to the nearer edge of the loaded area
h_c is the height of the wall to the level of the load
A_{ef} is the effective area of bearing (i.e. $l_{efm}t$)
l_{efm} is the effective length 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

$\frac{A_b}{A_{ef}}$ is not to be taken greater than 0.45.

Before considering some of the limitations upon N_{Rdc}, it is useful to look at some cases (Figure 6.8 and 6.9).

Figure 6.9. Graph showing the enhancement factor as given in Clause 6.1.3: Concentrated loads under bearings. (Data taken from BS EN 1996-1-1:2005) *Figure H.1*



The further clauses are all self-explanatory. However, three points are highlighted:

Clause 6.1.3(3)
Clause 6.1.3(4)
Clause 6.1.3(7)

- for Group 2, 3 and 4 units, β cannot be greater than 1 (i.e. $\beta \leq 1$)
- the limit for the eccentricity of load of $t/4$ permits the wall to be effectively loaded for only half of its thickness
- the maximum value possible for β is 1.5.

Annex H

A graphical method of calculating β is given in Annex H of EN 1996-1-1.

Note that the maximum value of β is 1.5; that is, the overstress allowed because the load is concentrated to $\leq$50%.

Clause 6.2
Clause 6.2(1)P

6.2. Unreinforced masonry walls subjected to shear loading

The principle is first established that the design value for the shear resistance of the wall shall exceed the design applied shear force:

Eqn 6.12

$$V_{\mathrm{Ed}} \leq V_{\mathrm{Rd}} \tag{6.13}$$

Clause 6.2(2)

The design value for shear resistance is given by the standard equation

Eqn 6.13

$$V_{\mathrm{Rd}} = f_{\mathrm{vd}} t l_{\mathrm{c}} \tag{6.14}$$

where

- f_{vd} is the design value of the shear strength of masonry, obtained from clauses 2.4.1 and 3.6.2, based on the average of the vertical stresses over the compressed part of the wall that is providing the shear resistance
- t is the wall thickness
- l_{c} is that portion of the wall which carries compressive stress (i.e. any length which is in tension is ignored).

The calculation of l_{c} should assume a linear stress distribution.

Clause 6.2(4)P

An important principle is laid down, namely that where webs join flanges to form either T sections or I beams, the shear at the junctions should be checked. This ensures that complex sections will retain their integrity and not break into a series of (much less stiff) sub-walls.

Finally, the compressed part of the wall should be checked to ensure that the sum of the dead loads applied together with the extra compressive stress due to wind load (under whichever wind combination is being considered) does not overstress the masonry in design terms.

Clause 6.3
Clause 6.1.3(1)

6.3. Unreinforced masonry walls subjected to lateral load

The principle is first established that the design applied moment within the masonry panel shall not exceed the design value for the moment of resistance of the panel:

Eqn 6.14

$$M_{\mathrm{Ed}} \leq M_{\mathrm{Rd}} \tag{6.15}$$

From here, designers familiar with the BS 5628 method for wind-loaded panel design will recognise that the method in Eurocode 6 has been adopted (virtually in its entirety) from BS 5628.

Clause 6.1.3(2)
Clause 6.1.3(3)

The orthogonal ratio (of strengths) μ has been retained, as has the familiar design moment of resistance:

Eqn 6.15

$$\mathrm{M}_{\mathrm{Rd}} = f_{\mathrm{xd}} Z \tag{6.16}$$

where

- f_{xd} is the design flexural strength appropriate to the plane of bending, obtained from clause 3.6.3, 6.3.1(4) or 6.6.2(9)
- Z is the elastic section modulus of unit height or length of the wall.

Clause 6.1.3(4)

Where it may be necessary, f_{xd1} (in the weak direction) can be modified by adding in the stress due to a vertical load. This could be the self-weight of (the appropriate portion of) the panel. This use of the self-weight stress of the panel is generally only done where it proves necessary: it is an easier task to justify the panel using the unmodified f_{xd1} when this gives a satisfactory outcome. (A modified f_{xd1} alters μ, and this in turn alters the bending moment coefficient α.)

It should be noted that where the dead weight stress is added to f_{xd1}, the reader may believe that σ_d is found by dividing the characteristic stress by γ_M. The intention, however, is to use the characteristic load multiplied by γ_f; the value of γ_f should be taken as for a favourable condition where the load/stress σ_d is favourable to the success of the member (i.e. σ_d should be underestimated not overestimated). This, in part, is due to the flexural strength f_{xd1} being already a design strength. The approach in BS 5628 was somewhat different, although the result is much the same.

$$f_{xd1,app} = f_{xd1} + \sigma_d \qquad (6.17)$$ *Eqn 6.16*

where

f_{xd1} is the design flexural strength of masonry with the plane of failure parallel to the bed joints (see clause 3.6.3)
σ_d is the design compressive stress on the wall, not greater than $0.2f_d$.

Note the limitation on the value of σ_d: this is not the method to use for a vertically loaded wall which also has a light wind load.

Simple rules are given to define the length of outstanding flanges that can be considered to form a structural section when calculating the section modulus of a pier in a wall. The section modulus would then apply to those sections which are formed from outstanding lengths that are the lesser of: *Clause 6.3.1(5)*

$\leq h/10$ (for vertically spanning walls)
$\leq h/5$ (for cantilever walls)
$\leq$half the clear distance between piers.

These are similar – if somewhat different in value – to the concept used in BS 5628, which was based on a $12t$ or $6t$ outstand length, depending on whether the outstand was continuous or a cantilever.

When designing cavity walls, the approach used in BS 5628 can still be used – namely adding together the design load resistance of both leaves. Presumably to acknowledge the correctness of the approach that load tends to be attracted in proportion to stiffness, a stiffness method can also be used. These alternatives may well give different solutions. It is also recognised that where a deep or unusual chase is present (beyond that which might be considered 'normal'), this requires to be taken into account. It is suggested that the reduced wall thickness be used at the chase or recess position. *Clause 6.1.3(6)*

6.3.2 Walls arching between supports

Clause 6.3.2
Clause 6.3.2(2)

A 'full arching' solution is permitted in Eurocode 6, as permitted in BS 5628. The method can only be used when a horizontal or vertical arch develops within the wall: this can readily happen where brick masonry is solidly built within (and confined by) a steel or concrete frame. Clay masonry has a long-term expansion and is, therefore, normally 'jammed' within the frame horizontally and vertically and remains so. With concrete and calcium silicate masonry, however, this rarely happens: instead, such walls shrink slightly, thus negating the whole concept of arch thrust. Although possible in theory, is horizontal arching more a concept than a reality with concrete and calcium silicate masonry built within frames. If the wall arches vertically, however, and is within a concrete frame that both shrinks and creeps vertically, it is possible to consider that a true arch with thrust can develop vertically.

Clause 6.3.2(4)
Clause 6.3.2(5)

The detail of the various clauses either follows logic or simple engineering mechanics.

The arch rise r is given by

$$r = 0.9t - d_a \qquad (6.18)$$ *Eqn 6.17*

Clause 6.2(5)

where

- t is the thickness of the wall, taking into account the reduction in thickness resulting from recessed joints
- d_a is the deflection of the arch under the design lateral load – it may be taken to be zero for walls having a length to thickness ratio of 25 or less.

The stress block providing the arch thrust is allowed to carry a level of stress equal to $1.5f_d$, to allow for the concentrated nature of the compressive stress derived from bending. f_d must, of course, be from the appropriate direction.

Clause 6.3.2(6)

The maximum design arch thrust, N_{ad}, is given by

Eqn 6.18

$$N_{ad} = 1.5f_d \frac{t}{10} \quad \text{(per unit length)} \tag{6.19}$$

Clause 6.3.2(6)

The value of the design lateral load than can be carried is given by

Eqn 6.19

$$q_{lat,d} = f_d \left(\frac{t}{l_a}\right)^2 \tag{6.20}$$

where

- N_{ad} is the design arch thrust
- $q_{lat,d}$ is the design lateral strength per unit area of wall
- t is the thickness of the wall
- f_d is the design compressive strength of the masonry in the direction of the arch thrust, obtained from clause 3.6.1
- l_a is the length or the height of the wall between supports capable of resisting the arch thrust

provided that:

- any damp-proof course or other plane of low frictional resistance in the wall can transmit the relevant horizontal forces
- the design value of the stress due to vertical load is not less than 0.1 N/mm^2
- the slenderness ratio does not exceed 20.

Clause 6.3.3

6.3.3 Walls subjected to wind loading

Little further comment to the favourable comment made elsewhere is required here. The method is essentially the BS 5628 method. Specimen calculations are given in Section A5.3 of this guide.

Clause 6.3.1(2)
Clause 6.3.1(3)
Clause 6.3.1(4)

The basic equation to satisfy (considering the horizontal direction of bending) is

$$W_{Ed}\alpha l^2 \leq f_{xd}Z \tag{6.21}$$

where

- W_{Ed} is the design value of the applied wind load
- α is the bending moment coefficient
- l is the length of the panel
- f_{xd} is the design flexural strength of masonry in the strong direction
- Z is the section modulus of the leaf being designed.

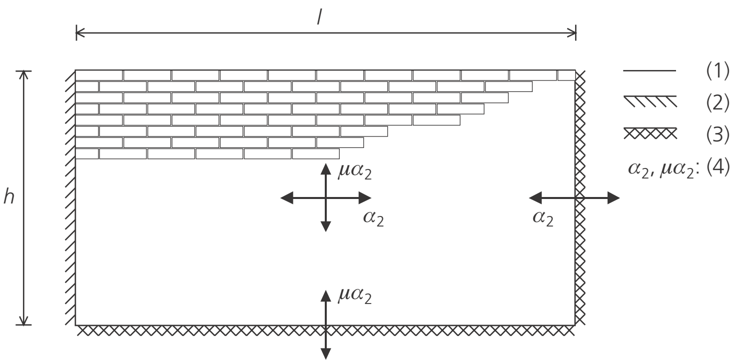
Table 6.1 lists bending moment coefficients in single-leaf laterally loaded wall panels with a thickness of less than 250 mm (see Figure 6.10 for panel edge supports).

Clause 5.5.5(6)

In the case of cavity walls, full continuity may be assumed even if only one leaf is continuously bonded across a support, provided that the cavity wall has adequate and proper wall ties.

The use of specimen calculations is illustrated in Section A5.3 of this guide.

Figure 6.10. Key to support conditions used in Table 6.1. (Reproduced from BS EN 1996-1-1 © British Standards Institution 2005) *Figure E.1*



Key
(1) Free edge
(2) Simply supported edge
(3) Fully restrained/continuous edge
(4) α_2, $\mu\alpha_2$: moment coefficients in the indicated directions

6.3.4 Walls subjected to lateral loading from earth and water

Clause 6.3.4

Although there is some use made of reinforced masonry earth-retaining walls, comparatively few retaining walls are constructed. Their use is permitted, however, and the method of design is similar to that used for wind-loaded panels. The 'weak-direction' flexural strength f_{xk1} should however be reduced to zero – thus forcing the design to be justified – by:

(*a*) horizontal spanning
(*b*) vertical arching or
(*c*) a combination of (*a*) and (*b*).

By and large, modern house construction does not involve the use of basements which are common in some European countries and very common in North America. A simplified design approach for the design of basement walls subjected to lateral earth pressure is given in the simple rules code EN 1996-3.

6.3.5 Walls subjected to lateral loading from accidental situations

Clause 6.3.5

There is little commentary to add to what has gone before. Masonry struggles to sustain heavy accidental overload applied locally. As mentioned previously, vertical arching action (when there is sufficient dead load applied per unit length to the wall) can be used to prove that the wall can be justified.

What is not often stated is the undoubted truth that when a local accidental event does cause local damage to part of a masonry structure, the indeterminacy of the structure as a whole tends to permit masonry structures to survive with less disproportionate damage to the cause: masonry is a forgiving material, with a reasonably good record for 'survivability'.

6.4. Unreinforced masonry walls subjected to combined vertical and lateral loading

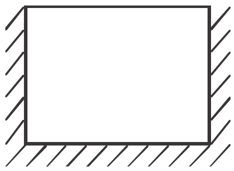
Clause 6.4

Three methods are offered by Eurocode 6, all of which can be supported by theoretical argument:

- a stability check with an additional eccentricity at mid-height, to allow for the wind moment
- using the wind panel design method and modifying the flexural strength in the weak direction (f_{xk1}) to account for the dead load stress
- a method that uses Annex I.

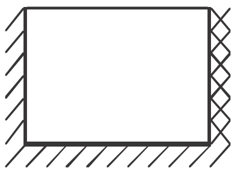
Table 6.1. Bending moment coefficients α_2 in single-leaf laterally loaded wall panels of thickness less than 250 mm. For complete data sets, see the tables in Annex E of BS EN 1996-1-1

Wall support condition A



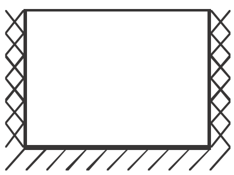
μ	*h/l*							
	0.30	0.50	0.75	1.00	1.25	1.50	1.75	2.00
1.00	0.031	0.045	0.059	0.071	0.079	0.085	0.090	0.094
0.90	0.032	0.047	0.061	0.073	0.081	0.087	0.092	
0.80	0.034	0.049	0.064	0.075	0.083	0.089		
0.70	0.035	0.051	0.066	0.077	0.085			
0.60	0.038	0.053	0.069	0.080				
0.50	0.040	0.056	0.073					
0.40	0.043	0.061						
0.35	0.045							
0.30								
0.25								
0.20								
0.15								
0.10								
0.05								

Wall support condition B



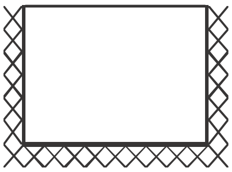
μ	*h/l*							
	0.30	0.50	0.75	1.00	1.25	1.50	1.75	2.00
1.00	0.024	0.035	0.046	0.053	0.059	0.062	0.065	0.068
0.90	0.025	0.036	0.047	0.055	0.060	0.063	0.066	
0.80	0.027	0.037	0.049	0.056	0.061	0.065		
0.70	0.028	0.039	0.051	0.058	0.062			
0.60	0.030	0.042	0.053	0.059				
0.50	0.031	0.044	0.055					
0.40	0.034	0.047						
0.35	0.035							
0.30								
0.25								
0.20								
0.15								
0.10								
0.05								

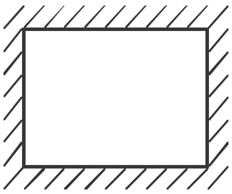
Wall support condition C



μ	*h/l*							
	0.30	0.50	0.75	1.00	1.25	1.50	1.75	2.00
1.00	0.020	0.028	0.037	0.042	0.045	0.048	0.050	0.051
0.90	0.021	0.029	0.038	0.043	0.046	0.048	0.050	
0.80	0.022	0.031	0.039	0.043	0.047	0.049		
0.70	0.023	0.032	0.040	0.044	0.048			
0.60	0.024	0.034	0.041	0.046				
0.50	0.025	0.035	0.043					
0.40	0.027	0.038						
0.35	0.029							
0.30								
0.25								
0.20								
0.15								
0.10								
0.05								

Table 6.1. Continued

Wall support condition D	μ	*h/l*							
		0.30	0.50	0.75	1.00	1.25	1.50	1.75	2.00
	1.00	0.031	0.021	0.029	0.035	0.040	0.043	0.045	0.047
	0.90	0.014	0.022	0.031	0.036	0.040	0.043	0.046	
	0.80	0.015	0.023	0.032	0.038	0.041	0.044		
	0.70	0.016	0.025	0.033	0.039	0.043			
	0.60	0.017	0.026	0.035	0.040				
	0.50	0.018	0.028	0.037					
	0.40	0.020	0.031						
	0.35	0.022							
	0.30								
	0.25								
	0.20								
	0.15								
	0.10								
	0.05								

Wall support condition E	μ	*h/l*							
		0.30	0.50	0.75	1.00	1.25	1.50	1.75	2.00
	1.00	0.008	0.018	0.030	0.042	0.051	0.059	0.066	0.071
	0.90	0.009	0.019	0.032	0.044	0.054	0.062	0.068	
	0.80	0.010	0.021	0.035	0.046	0.056	0.064		
	0.70	0.011	0.023	0.037	0.049	0.059			
	0.60	0.012	0.025	0.040	0.053	.			
	0.50	0.014	0.028	0.044					
	0.40	0.017	0.032						
	0.35	0.018							
	0.30								
	0.25								
	0.20								
	0.15								
	0.10								
	0.05								

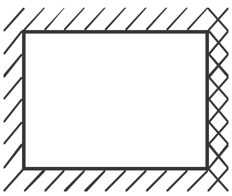
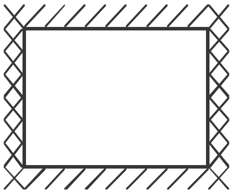
Wall support condition F	μ	*h/l*							
		0.30	0.50	0.75	1.00	1.25	1.50	1.75	2.00
	1.00	0.008	0.016	0.026	0.034	0.041	0.046	0.051	0.054
	0.90	0.008	0.017	0.027	0.036	0.042	0.048	0.052	
	0.80	0.009	0.018	0.029	0.037	0.044	0.049		
	0.70	0.010	0.020	0.031	0.039	0.046			
	0.60	0.011	0.022	0.033	0.042				
	0.50	0.013	0.024	0.036					
	0.40	0.015	0.027						
	0.35	0.016							
	0.30								
	0.25								
	0.20								
	0.15								
	0.10								
	0.05								

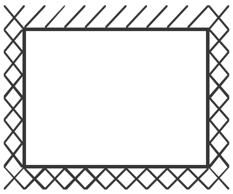
Table 6.1. Continued

Wall support condition G



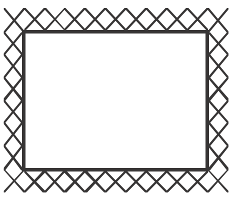
μ	h/l 0.30	0.50	0.75	1.00	1.25	1.50	1.75	2.00
1.00	0.007	0.014	0.022	0.028	0.033	0.037	0.040	0.042
0.90	0.008	0.015	0.023	0.029	0.034	0.038	0.041	
0.80	0.008	0.016	0.024	0.031	0.035	0.039		
0.70	0.009	0.017	0.026	0.032	0.037			
0.60	0.010	0.019	0.028	0.034				
0.50	0.011	0.021	0.030					
0.40	0.013	0.023						
0.35	0.014							
0.30								
0.25								
0.20								
0.15								
0.10								
0.05								

Wall support condition H



μ	h/l 0.30	0.50	0.75	1.00	1.25	1.50	1.75	2.00
1.00	0.005	0.011	0.018	0.024	0.029	0.033	0.036	0.039
0.90	0.006	0.012	0.019	0.025	0.030	0.034	0.037	
0.80	0.006	0.013	0.020	0.027	0.032	0.035		
0.70	0.007	0.014	0.022	0.028	0.033			
0.60	0.008	0.015	0.024	0.030				
0.50	0.009	0.017	0.025					
0.40	0.010	0.019						
0.35	0.011							
0.30								
0.25								
0.20								
0.15								
0.10								
0.05								

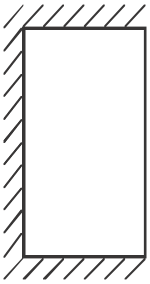
Wall support condition I



μ	h/l 0.30	0.50	0.75	1.00	1.25	1.50	1.75	2.00
1.00	0.004	0.009	0.015	0.021	0.026	0.030	0.033	0.036
0.90	0.004	0.010	0.016	0.022	0.027	0.031	0.034	
0.80	0.005	0.010	0.017	0.023	0.028	0.032		
0.70	0.005	0.011	0.019	0.025	0.030			
0.60	0.006	0.013	0.020	0.026				
0.50	0.007	0.014	0.022					
0.40	0.008	0.016						
0.35	0.009							
0.30								
0.25								
0.20								
0.15								
0.10								
0.05								

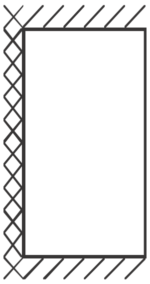
Table 6.1. Continued

Wall support condition J



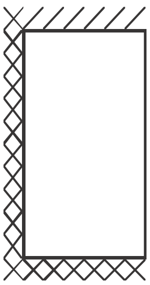
μ	*h/l*							
	0.30	0.50	0.75	1.00	1.25	1.50	1.75	2.00
1.00	0.009	0.023	0.046	0.071	0.096	0.122	0.151	0.180
0.90	0.010	0.026	0.050	0.076	0.103	0.131	0.162	
0.80	0.012	0.028	0.054	0.083	0.111	0.142		
0.70	0.013	0.032	0.060	0.091	0.121			
0.60	0.015	0.036	0.067	0.100				
0.50	0.018	0.042	0.077					
0.40	0.021	0.050						
0.35	0.024							
0.30								
0.25								
0.20								
0.15								
0.10								
0.05								

Wall support condition K



μ	*h/l*							
	0.30	0.50	0.75	1.00	1.25	1.50	1.75	2.00
1.00	0.009	0.021	0.038	0.056	0.074	0.091	0.108	0.123
0.90	0.010	0.023	0.041	0.060	0.079	0.097	0.113	
0.80	0.011	0.025	0.045	0.065	0.084	0.103		
0.70	0.012	0.028	0.049	0.070	0.091			
0.60	0.014	0.031	0.054	0.077				
0.50	0.016	0.035	0.061					
0.40	0.019	0.041						
0.35	0.021							
0.30								
0.25								
0.20								
0.15								
0.10								
0.05								

Wall support condition L



μ	*h/l*							
	0.30	0.50	0.75	1.00	1.25	1.50	1.75	2.00
1.00	0.006	0.015	0.029	0.044	0.059	0.073	0.088	0.102
0.90	0.007	0.017	0.032	0.047	0.063	0.078	0.093	
0.80	0.008	0.018	0.034	0.051	0.067	0.084		
0.70	0.009	0.021	0.038	0.056	0.073			
0.60	0.010	0.023	0.042	0.061				
0.50	0.012	0.027	0.048					
0.40	0.014	0.032						
0.35	0.016							
0.30								
0.25								
0.20								
0.15								
0.10								
0.05								

Clause 6.5

6.5. Ties

This section on ties quite properly addresses the basic issues that are implicit when ties are used to connect two different 'skins' or leaves. Having said that, the actual provisions are somewhat academic since ties, by and large, are governed more by 'hallowed custom and usage' than by detailed design.

It is a major issue if rigid wall ties (i.e. solid vertical twist-type ties) are used across the cavity between two leaves of dissimilar material. These could be, say, clay brickwork (which tends to expand with time) and either blockwork or calcium silicate or concrete brickwork (all of which tend to contract with time). Distress can result, including bowing of the wall. It is always better to use ties that can accommodate lateral (i.e. vertical shear) displacement. This allows for the use of different materials in the inner and outer leaves of cavity walls without distress.

While the clauses of Section 6.5 permit detailed examination and calculation of wall tie systems, it is the writer's experience that, with the possible exception of

- repairing damaged or faulty masonry buildings
- working in areas of high wind pressures,

the use of the minimum number of ties/m^2 of wall elevation normally adequately deals with the issue of strength.

Clause 6.6

6.6. Reinforced masonry members subjected to bending, bending and axial loading or axial loading

Clause 6.6.1

6.6.1 General

Clause 6.6.1(1)P
Clause 6.6.1(2)P

The principle is first set that the assumptions to be made in the design approach shall be as assumed in the clauses 6.6.1(1)P and 6.6.1(2)P. Other methods of fulfilling the assumptions are also listed.

Clause 6.6.2

6.6.2 Verification of reinforced masonry members subjected to bending and/or axial loading

Clause 6.6.2(1)P

The principle is set that the design load resistance of the member shall be equal to or greater than the design value of the applied load:

Eqn (6.21)

$$E_d \geq R_d \tag{6.22}$$

Clause 6.6.2(2)

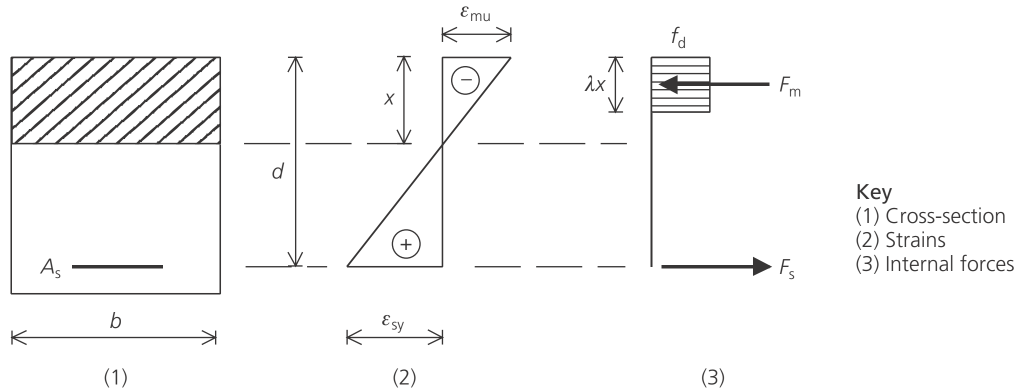
Having set the assumptions in clause 6.6.1 (above), the rules now call on the verification procedure to adopt these assumptions. In addition, the maximum tensile strain to be assumed in the reinforcement should be limited to 0.01.

Clause 6.6.2(4)

In a singly reinforced rectangular cross-section, a rectangular compressive stress distribution as shown in Figure 6.11 can be assumed.

Figure 6.4
Clause 6.6.2(3)

Figure 6.11. Stress and strain distribution. (Reproduced from BS EN 1996-1-1 © British Standards Institution 2005)



From consideration of the steel,

$$M_{\mathrm{Rd}} = A_{\mathrm{s}} f_{\mathrm{yd}} z \qquad (6.23)$$ *Eqn 6.22*

where z may be assumed to be, for a rectangular stress distribution,

$$z = d\left(1 - 0.5\frac{A_{\mathrm{s}} f_{\mathrm{yd}}}{b d f_{\mathrm{d}}}\right) \leq 0.95d \qquad (6.24)$$ *Eqn 6.23*

where

b is the width of the section
d is the effective depth of the section
A_{s} is the cross-sectional area of the reinforcement in tension
f_{d} is the design compressive strength of masonry in the direction of loading (obtained from clauses 2.4.1 and 3.6.1) or concrete infill (obtained from clauses 2.4.1 and 3.3), whichever is the lesser
f_{yd} is the design strength of reinforcing steel.

When considering the compression zone, *Clause 6.6.2(5)*

$M_{\mathrm{Rd}} \leq 0.4 f_{\mathrm{d}} b d^2$ for all Group 1 units except lightweight aggregate units (6.25a) *Eqn 6.24a*

$M_{\mathrm{Rd}} \leq 0.3 f_{\mathrm{d}} b d^2$ for all Group 2, 3, and 4 units and Group 1 lightweight aggregate units (6.25b) *Eqn 6.24b*

While the above is satisfactory information for, say, a small reinforced brickwork or blockwork lintel or beam, further rules are required for other reinforced masonry applications.

The width of a masonry section that is 'discreetly' reinforced should be limited to three times the overall thickness of the masonry. *Clause 6.6.2(6)*

Reinforced members carrying mainly compressive stress (of slenderness ratio greater than 12) may be designed using the method for unreinforced walls with the addition of an extra moment to account for second-order effects: *Clause 6.6.2(7)*

$$M_{\mathrm{ad}} = \frac{N_{\mathrm{Ed}} h_{\mathrm{ef}}^2}{2000t} \qquad (6.26)$$ *Eqn 6.25*

where

N_{Ed} is the design value of the vertical load
h_{ef} is the effective height of the wall
t is the thickness of the wall.

Rewritten in terms of the additional eccentricity, *Clause 6.6.2(8)*

$$\text{Additional eccentricity} = \frac{h_{\mathrm{ef}}^2}{2000t} \qquad (6.27)$$ *Eqn 6.26*

This additional eccentricity will have to be added algebraically with the correct sign.

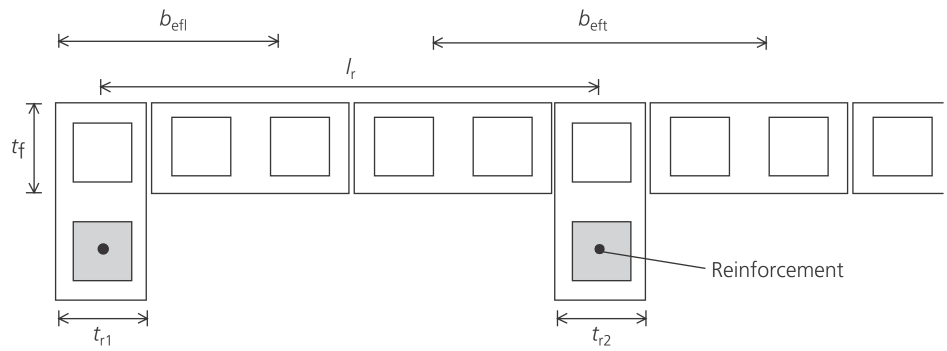
However, if the axial force is small (i.e. if $\sigma_{\mathrm{d}} \leq 0.3 f_{\mathrm{d}}$) it may be ignored, and the member designed solely for bending.

When bed-joint reinforcement is used to enhance the lateral strength of a masonry panel, a method is offered to first establish (and then use) an (enhanced) apparent (strong direction) f_{xd2}: *Clause 6.6.2(9)*

$$f_{\mathrm{xd2,app}} = \frac{6 A_{\mathrm{s}} f_{\mathrm{yd}} z}{t^2} \qquad (6.28)$$ *Eqn 6.27*

Figure 6.6

Figure 6.12. Effective width of flanges. (Reproduced from BS EN 1996-1-1 © British Standards Institution 2005)



When this has been found, it may be used with f_{xd1} (presumably the apparent value of this, $f_{xd1,app}$, will be used) to establish μ, then α, and then a value for design moment of resistance or the design applied load.

As the size of many masonry panels is being 'stretched' to ever greater dimensions, the use of bed-joint reinforcement is becoming more common.

Clause 6.6.3

6.6.3 Flanged reinforced members

The dimension rules for flanged members is dealt with first:

$$b_{efl} = \text{the lesser of} \begin{cases} t_{r1} + 6t_f \\ l_r/2 \\ h/6 \\ \text{actual width of the flange} \end{cases} \qquad b_{eft} = \text{the lesser of} \begin{cases} t_{r2} + 12t_f \\ l_r \\ h/3 \\ \text{actual width of the flange} \end{cases}$$

In addition to the rules given in Figure 6.12,

- t_f is the actual thickness of masonry in the flange $\leq 0.5d$ for the member
- the masonry will require to be checked to ensure that it spans between the points of lateral restraint (i.e. the points where the reinforcement is concentrated locally).

The design value for the moment resistance M_{Rd} can be calculated using the standard approach (Equation 6.22 of EN 1996-1-1). A second limit, however, also applies for the compressive zone of the section for flanged members:

Eqn 6.28

$$M_{Rd} \leq f_d b_{ef} t_f (d - 0.5t_f) \qquad (6.29)$$

Clause 6.6.4

6.6.4 Deep beams

M_{Rd} can be found using the standard equations check

$$M_{Rd} = A_s f_{yd} z$$

where z is taken as the lesser of

Eqn 6.29

$$z = 07l_{ef} \qquad (6.30)$$

or

Eqn 6.30

$$z = 0.4h + 0.2l_{ef} \qquad (6.31)$$

where

l_{ef} is the effective span of the beam.

Figure 6.13. Reinforcement of a deep beam. (Reproduced from BS EN 1996-1-1 © British Standards Institution 2005) *Figure 6.7*

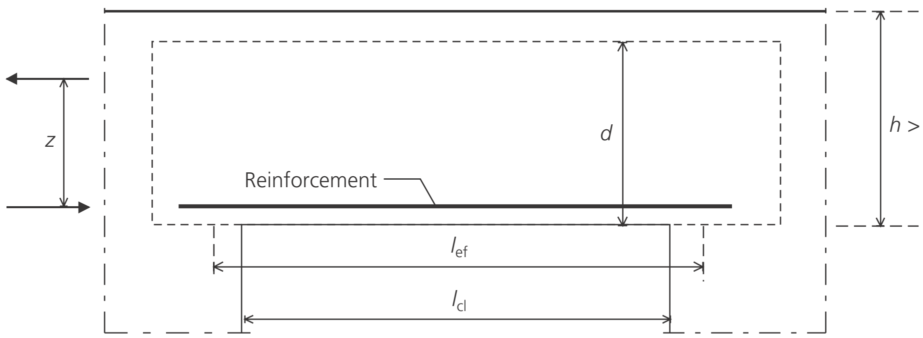


Figure 6.13 illustrates the reinforcement of a deep beam.

The design value of the moment of resistance, M_{Rd}, is

$M_{Rd} \leq 0.4 f_d b d^2$ for Group 1 units other than lightweight aggregate units (6.32a) *Eqn 6.31a*

and

$M_{Rd} \leq 0.3 f_d b d^2$ for Group 2, 3 and 4 units other than lightweight aggregate units (6.32b) *Eqn 6.31b*

where

b is the width of the beam
d is the effective depth of the beam, which may be taken as $1.3z$
f_d is the design compressive strength of the masonry.

Note that:

(*a*) f_d must be for the correct orientation of stress relative to the bed joint
(*b*) if concrete infill is present, f_d must be the lower value of the design compressive strength of the concrete infill or f_d from (*a*) above.

6.6.5 Composite lintels

Clause 6.6.5

The reader should be aware that an amendment is being prepared for this clause. Care and experience is required when using this clause as drafted.

6.7. Reinforced masonry members subjected to shear loading

Clause 6.7

6.7.1 General

Clause 6.7.1

The usual principle clause is present: *Clause 6.7.1(1)P*

$$V_{Ed} \leq V_{Rd} \quad (6.33)$$ *Eqn 6.32*

(see clause 8.2.3(5).) V_{Rd} may be calculated by either: *Clause 6.7.1(2)*

- ignoring the contribution of any shear reinforcement if the area of the shear reinforcement is less than 0.05% of the cross-sectional area of the member or
- taking the contribution of the shear reinforcement into account when the area of the shear reinforcement is greater than 0.05% of the cross-sectional area of the member.

The extent of any contribution of concrete infill to the shear resistance of the reinforced masonry member should be considered. *Clause 6.7.1(3)*

Where the concrete infill shear resistance contributes much more than the masonry, the designer should use EN 1992-1-1 and ignore the strength of the masonry.

Clause 6.7.2
Clause 6.7.2(1)
Clause 6.7.2(2)

6.7.2 Verification of reinforced masonry walls subjected to horizontal loads in the plane of the wall

For reinforced masonry walls containing vertical reinforcement,

Eqn 6.35
Eqn 6.33

$$V_{Ed} \leq V_{Rd1} + V_{Rd2} \tag{6.34}$$

where

V_{Rd1} is the design value of the shear resistance of unreinforced masonry,

Eqn 6.39

$$V_{Rd1} = f_{vd}bd \tag{6.35}$$

f_{vd} is the design shear strength of masonry
b is the minimum width of the beam over the effective depth
d is the effective depth of the beam.
V_{Rd2} is the design value of the contribution of the reinforcement, given by

Eqn 6.36

$$V_{Rd2} = 0.9A_{sw}f_{yd} \tag{6.36}$$

A_{sw} is the total area of the horizontal shear reinforcement over the part of the wall being considered
f_{yd} is the design strength of the reinforcing steel.

Clause 6.7.2(3)

Vertical reinforcement may not be considered, in which case $V_{Rd2} = 0$. Where shear reinforcement is taken into account,

Eqn 6.37

$$\frac{V_{Rd1} + V_{Rd2}}{tl} \leq 2.0\,\mathrm{N/mm^2} \tag{6.37}$$

where

t is the thickness of the wall
l is the length or, where appropriate, the height of the wall.

Clause 6.7.3
Clause 6.7.3(1)

6.7.3 Verification of reinforced masonry beams subjected to shear loading

When ignoring the contribution of any shear reinforcement,

Eqn 6.38

$$V_{Ed} \leq V_{Rd1} \tag{6.38}$$

where

Eqn 6.39

$$V_{Rd1} = f_{vd}bd \tag{6.39}$$

Annex J

In the case of walls or beams where the main reinforcement is placed in pockets, cores or cavities filled with concrete infill as described in clause 3.3, the value of f_{vd} used to calculate V_{Rdl} may be obtained from the following equation:

Eqn J.1

$$f_{vd} = \frac{0.35 + 17.5\rho}{\gamma_M} \tag{6.40}$$

provided that f_{vd} is not taken to be greater than $0.7\gamma_M/\mathrm{N/mm^2}$, where

Eqn J.2

$$\rho = \frac{A_s}{bd} \tag{6.41}$$

A_s is the cross-sectional area of the primary reinforcement
b is the width of the section
d is the effective depth
γ_M is the partial factor for masonry.

There was a misprint in Annex J when it was first issued. The '(2)' should not have been present – in other words, the complete annex should be merely a single clause, and the last six lines of Annex J should read:

For simply supported reinforced beams or cantilever retaining walls where the main reinforcement is placed in pockets, cores or cavities filled with concrete infill as described in 3.3 and where the ratio of the shear span, a_v, to the effective depth, d, is 6 or less, f_{vd} may be increased by a factor, χ, where:

$$\chi = \left[2.5 - 0.25\frac{a_v}{d}\right] \quad (6.42)$$

Eqn J.3

provided that f_{vd} is not taken to be greater than $1{,}75/\gamma_M$ N/mm^2.

The shear span, a_v, is taken to be the maximum bending moment in the section divided by the maximum shear force in the section.

(The reader should note that the wording above incorporates an amendment that is proposed but which has not yet taken effect at the time this guide was published. The above wording mirrors that in BS 5628, which was submitted to CEN but was inadvertently altered during final editing.)

Detailed guidance is given when shear reinforcement is taken into account for reinforced masonry beams.

Clause 6.7.3(3)
Clause 6.7.3(4)

Guidance is also offered for deep beams.

Clause 6.7.4

6.8. Prestressed masonry

Clause 6.8

Prestressed masonry is not dealt with in any great detail in EN 1996-1-1. It is, however, more generously covered in PD 6697 which of course relies on Eurocode 6 as the base document for loading conditions, material properties such as f_d, etc. and partial factors.

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Chapter 7
Serviceability limit state

This section needs little comment since most unreinforced masonry designed to meet the ultimate limit state will also meet the serviceability limit state. Similarly with reinforced masonry members, when they are sized to be within the limits given in Eurocode 6, their deflection should be acceptable. The remaining clauses are reasonably self-explanatory. Having said this, there are one or two things that will benefit from being highlighted.

7.1. General

The principle is first set that all masonry designed by Eurocode 6 should not exceed the serviceability limit state. *Clause 7.1(1)P*

7.2. Unreinforced masonry walls

For unreinforced laterally loaded masonry walls there is the requirement for them to satisfy a 'size limit' effectively given by graphs in Annex F of EN 1996-1-1 (this annex may be used in the UK). This equates to the two conditions that were given in BS 5628: *Clause 7.2(5)* *Annex F*

- a limit on the elevation area being less than nt_{ef}^2, where n is a number that depends on the number of supported sides and on the type of edge support
- an upper limit of $50t_{ef}$ is given for both the length and the height of the panel.

Both of these limits are subsumed into Annex F, and there are no numerical limits in Eurocode 6 that correspond to the limits from BS 5628 given above. *Annex F*

As mentioned earlier, there is some concern at the large panel sizes that can pass the Annex F test. Designers should therefore use judgement for panels that seem to them to be larger than normal but which still pass the requirements of Annex F. Current thinking (which may yet appear in some form of amendment to the National Annex) is that the $50t_{ef}$ is still a useful limit on the length and height dimension of a panel. The designer may wish to consider the dimensions of the panel being designed bearing this in mind, as well as Annex F.

7.3. Reinforced masonry members

When calculating the deflection of reinforced masonry members using the modulus of elasticity, the long-term modulus should be used: *Clause 7.3.3*

$$E_{\text{long term}} = \frac{E}{1 + \phi_\infty} \tag{7.1}$$

Eqn 3.8

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Chapter 8
Detailing

As with Section 7, this section requires little comment since most of the points covered, if not known to the designer, are fairly self-explanatory. Having said this, the reader's attention is drawn to clause 8.5. *Clause 8.5*

8.5. Connection of walls *Clause 8.5*

- This takes on greater importance than in the past due to the new design clauses – which were not part of BS 5628 – that permit the effective height of a wall to be modified if one or both of the vertical wall edges is supported. *Clause 8.5.2*
- For the connection of floors and roofs to load-bearing masonry walls, the designer should see specimen details in PD 6697. These are similar to those contained in BS 5628. *Clause 8.5.1.2*
- The number of ties required (per square metre on elevation) to connect the leaves of *Clause 8.5.2.2*
  – cavity and veneer walls *Clause 8.5.2.3*
  – double-leaf walls
  is designated as an NDP. The number of ties required for both situations is given as 2.5 in the UK National Annex. *Clause NA.2.16* *Clause NA.2.17*

The reader's attention is similarly drawn to clause 8.6. *Clause 8.6*

8.6. Chases and recesses on walls

New detailed guidance is offered here to define the extent of acceptable chases. The values are NDPs and the designer should therefore consult the National Annex. *Clause NA.2.18* *Clause NA.2.19*

Minimum thickness of wall

Finally, and mainly for the sake of completeness, the minimum wall thickness that can be designed using Eurocode 6 is given in clause 8.1.2. For a single-leaf wall and the leaf of a cavity wall, these minimum thicknesses are given as 90 and 75 mm, respectively. *Clause 8.1.2* *Clause NA.2.15*

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Annex 1
Characteristic compressive strength of masonry f_k based on the UK National Annex

A1.1. General-purpose mortar: bricks

Without longitudinal joints

Group 1, clay

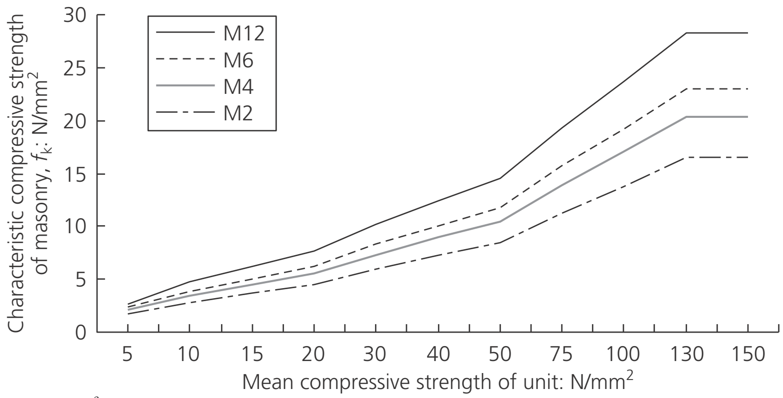
Wall without a longitudinal joint: constructed with general-purpose mortar and with standard-format clay bricks having no more than 25% of formed voids or 20% frogs; Group 1 units. See Table A1.1(a) and Figure A1.1(a).

Table A1.1(a). Wall without a longitudinal joint: 65 mm high × 102 mm thick standard format Group 1 bricks

$\delta = 0.85$ $K = 0.50$ (i.e. wall thickness = brick thickness)
Mortar Mean compressive strength of unit (N/mm^2) to BS EN 771-1 (not normalised)

UK designation	EN 1996-1-1	5	10	15	20	30	40	50	75	100	130	150
(i)	M12	2.6	4.7	6.3	7.7	10.2	12.4	14.5	19.3	23.6	28.3	28.3
(ii)	M6	2.4	3.8	5.1	6.2	8.3	10.1	11.8	15.7	19.2	23.0	23.0
(iii)	M4	2.1	3.4	4.5	5.5	7.3	8.9	10.5	13.9	17.0	20.3	20.3
(iv)	M2	1.7	2.8	3.7	4.5	5.9	7.3	8.5	11.3	13.8	16.5	16.5

Figure A1.1(a). Wall without a longitudinal joint: 65 mm high × 102 mm thick standard format Group 1 bricks



$f_k = K f_b^{\alpha} f_m^{\beta}$

f_k is the characteristic strength of masonry
K is a constant which may be taken from the National Annex
f_b is the normalised compressive strength of the units
f_m is the compressive strength of the mortar
α, β are constants which may be taken from the National Annex

f_b = CF × δ × mean compressive strength for bricks and blocks

CF is a conditioning factor

Group 1, concrete

Wall without a longitudinal joint: constructed with general-purpose mortar and with standard-format concrete bricks having no more than 25% of formed voids or 20% frogs; Group 1 units. See Table A1.1(b) and Figure A1.1(b).

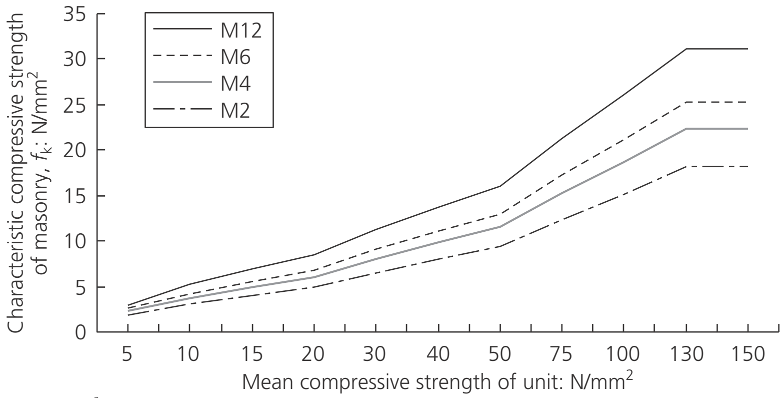
Table A1.1(b). Wall without a longitudinal joint: 65 mm high × 102 mm thick standard format Group 1 bricks

$\delta = 0.85$ $K = 0.55$ (i.e. wall thickness = brick thickness)

Mortar Mean compressive strength of unit (N/mm^2) to BS EN 771-3 (not normalised)

UK designation	EN 1996-1-1	5	10	15	20	30	40	50	75	100	130	150
(i)	M12	2.9	5.2	6.9	8.4	11.2	13.7	16.0	21.2	26.0	31.1	31.1
(ii)	M6	2.6	4.2	5.6	6.8	9.1	11.1	13.0	17.3	21.1	25.3	25.3
(iii)	M4	2.3	3.7	5.0	6.1	8.0	9.8	11.5	15.3	18.7	22.4	22.4
(iv)	M2	1.9	3.0	4.0	4.9	6.5	8.0	9.3	12.4	15.2	18.2	18.2

Figure A1.1(b). Wall without a longitudinal joint: 65 mm high × 102 mm thick standard format Group 1 bricks



$f_k = K f_b^{\alpha} f_m^{\beta}$

- f_k is the characteristic strength of masonry
- K is a constant which may be taken from the National Annex
- f_b is the normalised compressive strength of the units
- f_m is the compressive strength of the mortar
- α, β are constants which may be taken from the National Annex

f_b = CF × δ × mean compressive strength for bricks and blocks

CF is a conditioning factor

Group 1, calcium silicate

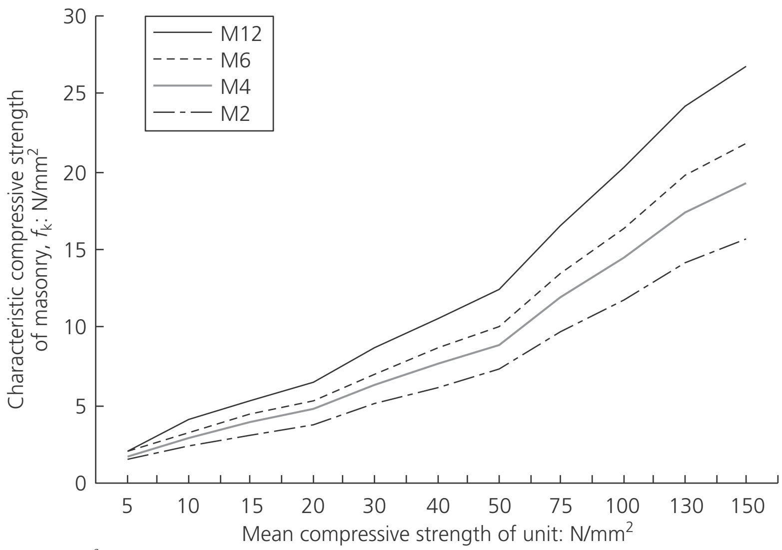
Wall without a longitudinal joint: constructed with general-purpose mortar and standard-format calcium silicate bricks having no more than 25% of formed voids or 20% frogs; Group 1 units. See Table A1.1(c) and Figure A1.1(c).

Table A1.1(c). Wall without a longitudinal joint: 65 mm high × 102 mm thick standard format Group 1 calcium silicate bricks

$\delta = 0.85$ $K = 0.50$ (i.e. wall thickness = brick thickness)
Mortar Mean compressive strength of unit (N/mm^2) to BS EN 771-2 (not normalised)

UK designation	EN 1996-1-1	5	10	15	20	30	40	50	75	100	130	150
(i)	M12	2.1	4.0	5.4	6.5	8.7	10.6	12.4	16.5	20.2	24.3	26.8
(ii)	M6	2.0	3.3	4.3	5.3	7.1	8.6	10.1	13.4	16.4	19.7	21.8
(iii)	M4	1.8	2.9	3.9	4.7	6.3	7.7	8.9	11.9	14.5	17.5	19.3
(iv)	M2	1.4	2.4	3.1	3.8	5.1	6.2	7.3	9.7	11.8	14.2	15.7

Figure A1.1(c). Wall without a longitudinal joint: 65 mm high × 102 mm thick standard format Group 1 calcium silicate bricks



$f_k = K f_b^{\alpha} f_m^{\beta}$

f_k is the characteristic strength of masonry
K is a constant which may be taken from the National Annex
f_b is the normalised compressive strength of the units
f_m is the compressive strength of the mortar
α, β are constants which may be taken from the National Annex

f_b = CF × δ × mean compressive strength for bricks and blocks

CF is a conditioning factor

See the section on calcium silicate units at the end of this annex.

Group 2, clay

Wall without a longitudinal joint: constructed with general-purpose mortar and with standard-format clay bricks having more than 25% voids but no more than 55% of formed voids; Group 2 units. See Table A1.1(d) and Figure A1.1(d).

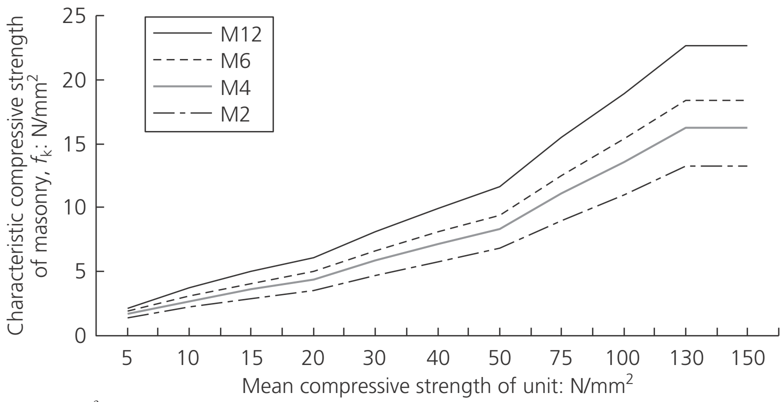
Table A1.1(d). Wall without a longitudinal joint: 65 mm high × 102 mm thick standard format Group 2 bricks

$\delta = 0.85$ $K = 0.40$ (i.e. wall thickness = brick thickness)

Mortar Mean compressive strength of unit (N/mm^2) to BS EN 771-1 (not normalised)

UK designation	EN 1996-1-1	5	10	15	20	30	40	50	75	100	130	150
(i)	M12	2.1	3.8	5.0	6.1	8.1	10.0	11.6	15.5	18.9	22.6	22.6
(ii)	M6	1.9	3.1	4.1	5.0	6.6	8.1	9.4	12.5	15.3	18.4	18.4
(iii)	M4	1.7	2.7	3.6	4.4	5.9	7.2	8.4	11.1	13.6	16.3	16.3
(iv)	M2	1.4	2.2	2.9	3.6	4.8	5.8	6.8	9.0	11.0	13.2	13.2

Figure A1.1(d). Wall without a longitudinal joint: 65 mm high × 102 mm thick standard format Group 2 bricks



$f_k = K f_b^{\alpha} f_m^{\beta}$

f_k is the characteristic strength of masonry
K is a constant which may be taken from the National Annex
f_b is the normalised compressive strength of the units
f_m is the compressive strength of the mortar
α, β are constants which may be taken from the National Annex

$f_b = \text{CF} \times \delta \times$ mean compressive strength for bricks and blocks

CF is a conditioning factor

Group 2, concrete

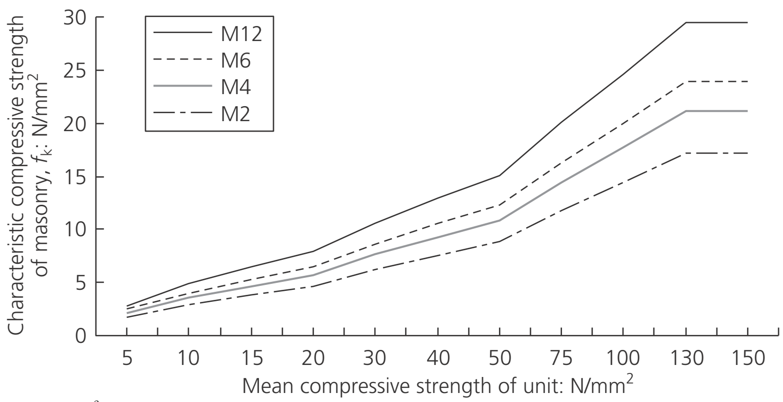
Wall without a longitudinal joint: constructed with general-purpose mortar and with standard-format concrete bricks having more than 25% voids but no more than 55% of formed voids; Group 2 units. See Table A1.1(e) and Figure A1.1(e).

Table A1.1(e). Wall without a longitudinal joint: 65 mm high × 102 mm thick standard format Group 2 concrete bricks

$\delta = 0.85$ $K = 0.52$ (i.e. wall thickness = brick thickness)
Mortar Mean compressive strength of unit (N/mm^2) to BS EN 771-3 (not normalised)

UK designation	EN 1996-1-1	5	10	15	20	30	40	50	75	100	130	150
(i)	M12	2.7	4.9	6.5	8.0	10.6	12.9	15.1	20.1	24.6	29.4	29.4
(ii)	M6	2.5	4.0	5.3	6.5	8.6	10.5	12.3	16.3	20.0	23.9	23.9
(iii)	M4	2.2	3.5	4.7	5.7	7.6	9.3	10.9	14.4	17.7	21.2	21.2
(iv)	M2	1.8	2.9	3.8	4.7	6.2	7.6	8.8	11.7	14.4	17.2	17.2

Figure A1.1(e). Wall without a longitudinal joint: 65 mm high × 102 mm thick standard format Group 2 concrete bricks



$f_k = K f_b^{\alpha} f_m^{\beta}$

f_k is the characteristic strength of masonry
K is a constant which may be taken from the National Annex
f_b is the normalised compressive strength of the units
f_m is the compressive strength of the mortar
α, β are constants which may be taken from the National Annex

f_b = CF × δ × mean compressive strength for bricks and blocks

CF is a conditioning factor

Group 2, calcium silicate

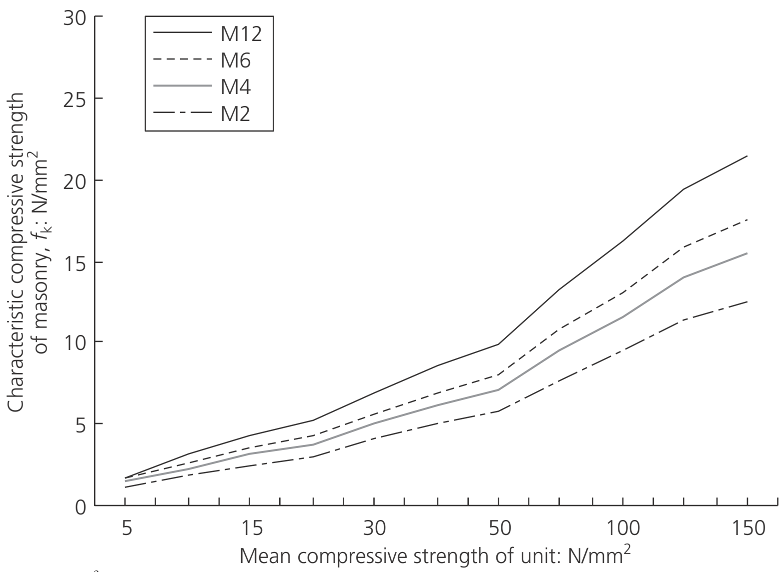
Wall without a longitudinal joint: constructed with general-purpose mortar and standard-format calcium silicate bricks having more than 25% voids but no more than 55% of formed voids; Group 2 units. See Table A1.1(f) and Figure A1.1(f).

Table A1.1(f). Wall without a longitudinal joint: 65 mm high × 102 mm thick standard format Group 2 calcium silicate bricks

$\delta = 0.85$ $K = 0.40$ (i.e. wall thickness = brick thickness)
Mortar Mean compressive strength of unit (N/mm^2) to BS EN 771-2 (not normalised)

UK designation	EN 1996-1-1	5	10	15	20	30	40	50	75	100	130	150
(i)	M12	1.6	3.2	4.3	5.2	7.0	8.5	10.0	13.2	16.2	19.4	21.5
(ii)	M6	1.6	2.6	3.5	4.3	5.7	6.9	8.1	10.7	13.1	15.8	17.4
(iii)	M4	1.4	2.3	3.1	3.8	5.0	6.1	7.2	9.5	11.6	14.0	15.4
(iv)	M2	1.2	1.9	2.5	3.1	4.1	5.0	5.8	7.7	9.4	11.3	12.5

Figure A1.1(f). Wall without a longitudinal joint: 65 mm high × 102 mm thick standard format Group 2 calcium silicate bricks



$f_k = K f_b^{\alpha} f_m^{\beta}$

f_k is the characteristic strength of masonry
K is a constant which may be taken from the National Annex
f_b is the normalised compressive strength of the units
f_m is the compressive strength of the mortar
α, β are constants which may be taken from the National Annex

f_b = CF × δ × mean compressive strength for bricks and blocks

CF is a conditioning factor

See the section on calcium silicate units at the end of this annex.

Group 3

Wall without a longitudinal joint. Group 3 clay units have not traditionally been used in the UK, so no values are available. Group 3 calcium silicate units are not used in Europe. Group 3 concrete units have not traditionally been used in the UK, so no values are available.

Group 4

Wall without a longitudinal joint. Group 4 clay units have not traditionally been used in the UK, so no values are available. Group 4 calcium silicate units are not used in Europe. Group 4 concrete units have not traditionally been used in the UK, so no values are available.

With longitudinal joints

Group 1, clay

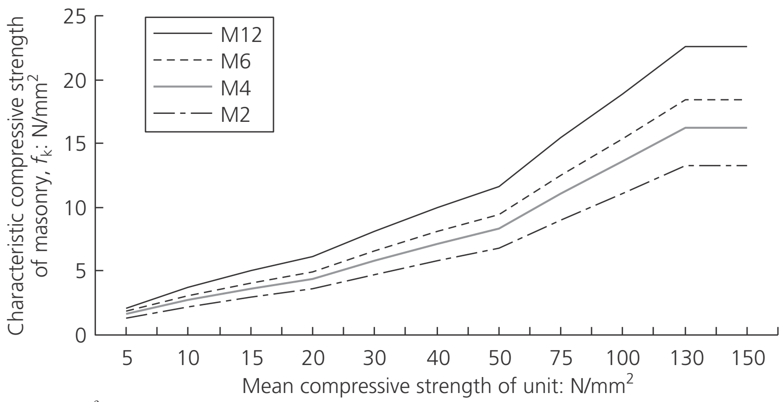
Wall with a longitudinal joint: constructed with general-purpose mortar and with standard-format clay bricks having no more than 25% of formed voids or 20% frogs; Group 1 units. See Table A1.1(g) and Figure A1.1(g).

Table A1.1(g). Wall with a longitudinal joint: 65 mm high × 102 mm thick standard format Group 1 bricks

$\delta = 0.85$ $K = 0.8 \times 0.50$ (i.e. wall thickness > brick thickness)
Mortar Mean compressive strength of unit (N/mm^2) to BS EN 771-1 (not normalised)

UK designation	EN 1996-1-1	5	10	15	20	30	40	50	75	100	130	150
(i)	M12	2.1	3.8	5.0	6.1	8.1	10.0	11.6	15.5	18.9	22.6	22.6
(ii)	M6	1.9	3.1	4.1	5.0	6.6	8.1	9.4	12.5	15.3	18.4	18.4
(iii)	M4	1.7	2.7	3.6	4.4	5.9	7.2	8.4	11.1	13.6	16.3	16.3
(iv)	M2	1.4	2.2	2.9	3.6	4.8	5.8	6.8	9.0	11.0	13.2	13.2

Figure A1.1(g). Wall without a longitudinal joint: 65 mm high × 102 mm thick standard format Group 1 bricks



$f_k = K f_b^{\alpha} f_m^{\beta}$

f_k is the characteristic strength of masonry
K is a constant which may be taken from the National Annex
f_b is the normalised compressive strength of the units
f_m is the compressive strength of the mortar
α, β are constants which may be taken from the National Annex

f_b = CF × δ × mean compressive strength for bricks and blocks

CF is a conditioning factor

Group 1, concrete

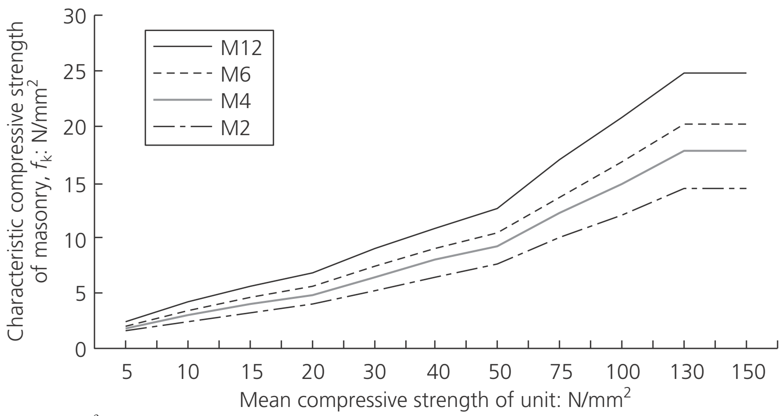
Wall with a longitudinal joint: constructed with general-purpose mortar and with standard-format concrete bricks having no more than 25% of formed voids or 20% frogs; Group 1 units. See Table A1.1(h) and Figure A1.1(h).

Table A1.1(h). Wall with a longitudinal joint: 65 mm high × 102 mm thick standard format Group 1 concrete bricks

$\delta = 0.85$ $K = 0.8 \times 0.55$ (i.e. wall thickness > brick thickness)
Mortar Mean compressive strength of unit (N/mm^2) to BS EN 771-3 (not normalised)

UK designation	EN 1996-1-1	5	10	15	20	30	40	50	75	100	130	150
(i)	M12	2.3	4.1	5.5	6.7	8.9	10.9	12.8	17.0	20.8	24.9	24.9
(ii)	M6	2.1	3.4	4.5	5.5	7.3	8.9	10.4	13.8	16.9	20.2	20.2
(iii)	M4	1.8	3.0	4.0	4.8	6.4	7.9	9.2	12.2	15.0	17.9	17.9
(iv)	M2	1.5	2.4	3.2	3.9	5.2	6.4	7.5	9.9	12.1	14.5	14.5

Figure A1.1(h). Wall with a longitudinal joint: 65 mm high × 102 mm thick standard format Group 1 concrete bricks



$f_k = K f_b^{\alpha} f_m^{\beta}$

f_k is the characteristic strength of masonry
K is a constant which may be taken from the National Annex
f_b is the normalised compressive strength of the units
f_m is the compressive strength of the mortar
α, β are constants which may be taken from the National Annex

f_b = CF × δ × mean compressive strength for bricks and blocks

CF is a conditioning factor

Group 1, calcium silicate

Wall with a longitudinal joint: constructed with general-purpose mortar and standard-format calcium silicate bricks having no more than 25% of formed voids or 20% frogs; Group 1 units. See Table A1.1(i) and Figure A1.1(i).

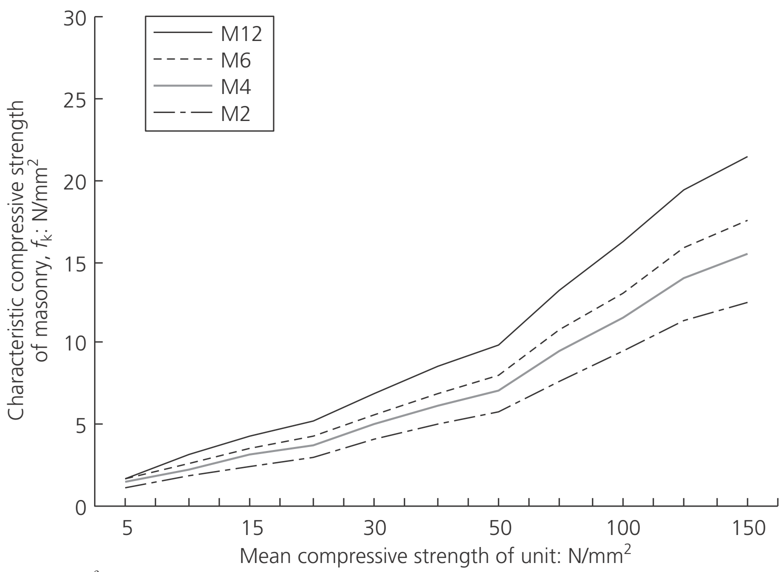
Table A1.1(i). Wall with a longitudinal joint: 65 mm high × 102 mm thick standard format Group 1 calcium silicate bricks

$\delta = 0.85$ $K = 0.50$ (i.e. wall thickness = brick thickness)

Mortar Mean compressive strength of unit (N/mm^2) to BS EN 771-2 (not normalised)

UK designation	EN 1996-1-1	5	10	15	20	30	40	50	75	100	130	150
(i)	M12	1.6	3.2	4.3	5.2	7.0	8.5	10.0	13.2	16.2	19.4	21.5
(ii)	M6	1.6	2.6	3.5	4.3	5.7	6.9	8.1	10.7	13.1	15.8	17.4
(iii)	M4	1.4	2.3	3.1	3.8	5.0	6.1	7.2	9.5	11.6	14.0	15.4
(iv)	M2	1.2	1.9	2.5	3.1	4.1	5.0	5.8	7.7	9.4	11.3	12.5

Figure A1.1(i). Wall with a longitudinal joint: 65 mm high × 102 mm thick standard format Group 1 calcium silicate bricks



$f_k = K f_b^{\alpha} f_m^{\beta}$

f_k is the characteristic strength of masonry
K is a constant which may be taken from the National Annex
f_b is the normalised compressive strength of the units
f_m is the compressive strength of the mortar
α, β are constants which may be taken from the National Annex

f_b = CF × δ × mean compressive strength for bricks and blocks

CF is a conditioning factor

See the section on calcium silicate units at the end of this annex.

Group 2, clay

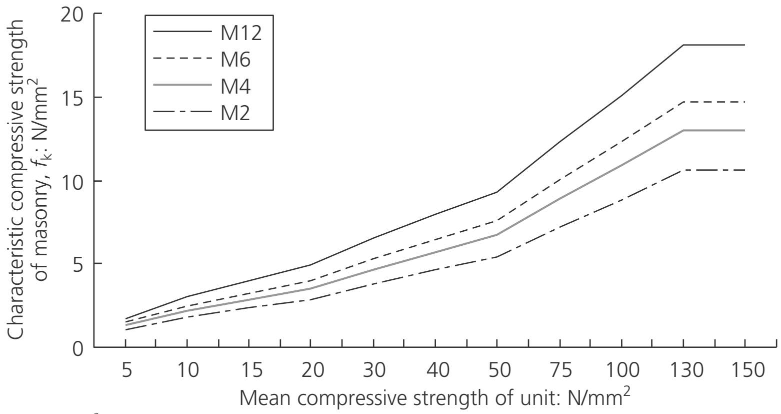
Wall with a longitudinal joint: constructed with general-purpose mortar and with standard-format clay bricks having more than 25% voids but no more than 55% of formed voids; Group 2 units. See Table A1.1(j) and Figure A1.1(j).

Table A1.1(j). Wall with a longitudinal joint: 65 mm high × 102 mm thick standard format Group 2 bricks

$\delta = 0.85$ $K = 0.8 \times 0.40$ (i.e. wall thickness > brick thickness)
Mortar Mean compressive strength of unit (N/mm^2) to BS EN 771-1 (not normalised)

UK designation	EN 1996-1-1	5	10	15	20	30	40	50	75	100	130	150
(i)	M12	1.7	3.0	4.0	4.9	6.5	8.0	9.3	12.4	15.1	18.1	18.1
(ii)	M6	1.5	2.5	3.3	4.0	5.3	6.5	7.6	10.0	12.3	14.7	14.7
(iii)	M4	1.3	2.2	2.9	3.5	4.7	5.7	6.7	8.9	10.9	13.0	13.0
(iv)	M2	1.1	1.8	2.3	2.9	3.8	4.7	5.4	7.2	8.8	10.6	10.6

Figure A1.1(j). Wall with a longitudinal joint: 65 mm high × 102 mm thick standard format Group 2 bricks



$f_k = K f_b^{\alpha} f_m^{\beta}$

f_k is the characteristic strength of masonry
K is a constant which may be taken from the National Annex
f_b is the normalised compressive strength of the units
f_m is the compressive strength of the mortar
α, β are constants which may be taken from the National Annex

f_b = CF × δ × mean compressive strength for bricks and blocks

CF is a conditioning factor

Group 2, concrete

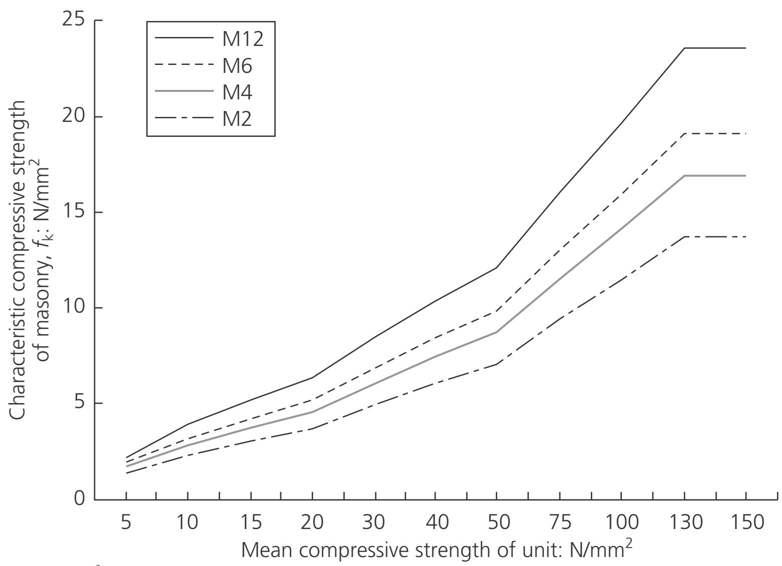
Wall with a longitudinal joint: constructed with general-purpose mortar and with standard-format concrete bricks having more than 25% voids but no more than 55% of formed voids; Group 2 units. See Table A1.1(k) and Figure A1.1(k).

Table A1.1(k). Wall with a longitudinal joint: 65 mm high × 102 mm thick standard format Group 2 concrete bricks

$\delta = 0.85$ $K = 0.8 \times 0.52$ (i.e. wall thickness > brick thickness)
Mortar Mean compressive strength of unit (N/mm^2) to BS EN 771-3 (not normalised)

UK designation	EN 1996-1-1	5	10	15	20	30	40	50	75	100	130	150
(i)	M12	2.2	3.9	5.2	6.4	8.5	10.3	12.1	16.1	19.7	23.5	23.5
(ii)	M6	2.0	3.2	4.2	5.2	6.9	8.4	9.8	13.1	16.0	19.1	19.1
(iii)	M4	1.7	2.8	3.7	4.6	6.1	7.4	8.7	11.6	14.1	16.9	16.9
(iv)	M2	1.4	2.3	3.0	3.7	4.9	6.0	7.1	9.4	11.5	13.8	13.8

Figure A1.1(k). Wall with a longitudinal joint: 65 mm high × 102 mm thick standard format Group 2 concrete bricks



$f_k = K f_b^{\alpha} f_m^{\beta}$

f_k is the characteristic strength of masonry
K is a constant which may be taken from the National Annex
f_b is the normalised compressive strength of the units
f_m is the compressive strength of the mortar
α, β are constants which may be taken from the National Annex

f_b = CF × δ × mean compressive strength for bricks and blocks

CF is a conditioning factor

Group 2, calcium silicate

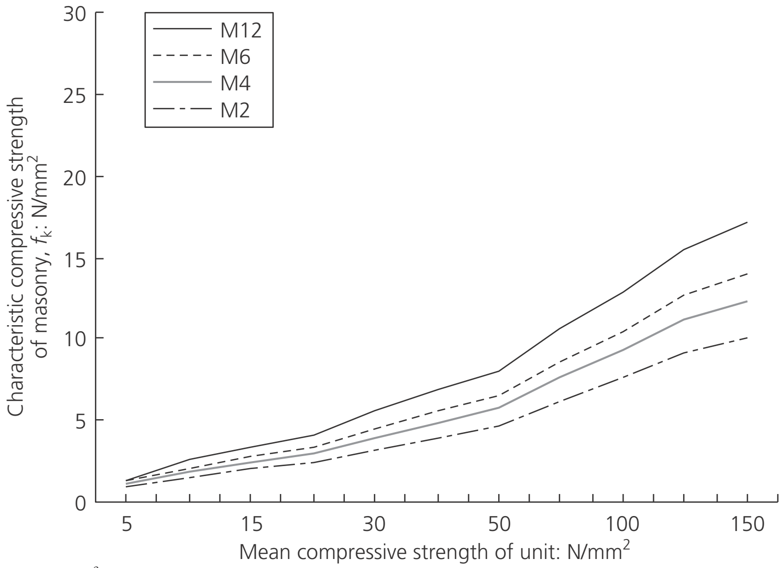
See Table A1.1(l) and Figure A1.1(l).

Table A1.1(l). Wall with a longitudinal joint: 65 mm high × 102 mm thick standard format Group 2 calcium silicate bricks

$\delta = 0.85$ $K = 0.40$ (i.e. wall thickness = brick thickness)
Mortar Mean compressive strength of unit (N/mm^2) to BS EN 771-2 (not normalised)

UK designation	EN 1996-1-1	5	10	15	20	30	40	50	75	100	130	150
(i)	M12	1.3	2.6	3.4	4.2	5.6	6.8	8.0	10.6	12.9	15.5	17.2
(ii)	M6	1.3	2.1	2.8	3.4	4.5	5.5	6.5	8.6	10.5	12.6	14.0
(iii)	M4	1.1	1.9	2.5	3.0	4.0	4.9	5.7	7.6	9.3	11.2	12.4
(iv)	M2	0.9	1.5	2.0	2.4	3.3	4.0	4.7	6.2	7.6	9.1	10.0

Figure A1.1(l). Wall with a longitudinal joint: 65 mm high × 102 mm thick standard format Group 2 calcium silicate bricks



$f_k = K f_b^{\alpha} f_m^{\beta}$

f_k is the characteristic strength of masonry
K is a constant which may be taken from the National Annex
f_b is the normalised compressive strength of the units
f_m is the compressive strength of the mortar
α, β are constants which may be taken from the National Annex

f_b = CF × δ × mean compressive strength for bricks and blocks

CF is a conditioning factor

See the section on calcium silicate units at the end of this annex.

Group 3

Wall with a longitudinal joint. Group 3 clay units have not traditionally been used in the UK, so no values are available. Group 3 calcium silicate units are not used in Europe. Group 3 concrete units have not traditionally been used in the UK, so no values are available.

Group 4

Wall with a longitudinal joint. Group 4 clay units have not traditionally been used in the UK, so no values are available. Group 4 calcium silicate units are not used in Europe. Group 4 concrete units have not traditionally been used in the UK, so no values are available.

A1.2. General-purpose mortar: blocks

Without longitudinal joints

Group 1, 100 mm autoclaved concrete or concrete aggregate

Wall without a longitudinal joint: constructed with general-purpose mortar and with standard-format aggregate concrete or autoclaved concrete blocks 215 mm high × 100 mm thick; Group 1 units. See Table A1.2(a) and Figure A1.2(a).

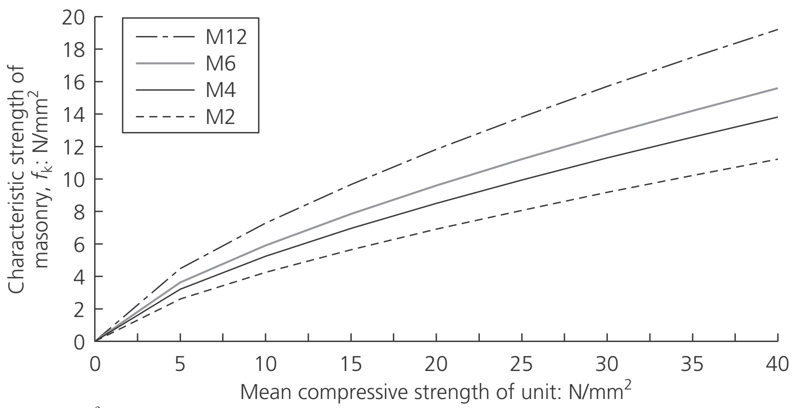
Table A1.2(a). Wall with a longitudinal joint: 215 mm high × 100 mm thick Group 1 blocks

$\delta = 1.38$ $K = 0.55$ (i.e. wall thickness = block thickness)

Mortar Mean compressive strength of unit (N/mm^2) to BS EN 771-3 or 4 (not normalised)

UK designation	EN 1996-1-1	2.9	3.6	5.2	7.3	10.4	17.5	22.5	30.0	40.0
(i)	M12	2.7	3.4	4.6	5.8	7.5	10.8	12.8	15.7	19.2
(ii)	M6	2.5	2.9	3.7	4.7	6.1	8.7	10.4	12.8	15.6
(iii)	M4	2.2	2.6	3.3	4.2	5.4	7.7	9.2	11.3	13.8
(iv)	M2	1.8	2.1	2.7	3.4	4.4	6.3	7.5	9.2	11.2

Figure A1.2(a). Wall without a longitudinal joint: 215 mm high × 100 mm thick Group 1 blocks



$f_k = K f_b^{\alpha} f_m^{\beta}$

f_k is the characteristic strength of masonry
K is a constant which may be taken from the National Annex
f_b is the normalised compressive strength of the units
f_m is the compressive strength of the mortar
α, β are constants which may be taken from the National Annex

f_b = CF × δ × mean compressive strength for bricks and blocks

CF is a conditioning factor

Group 1, 200 mm autoclaved concrete or concrete aggregate

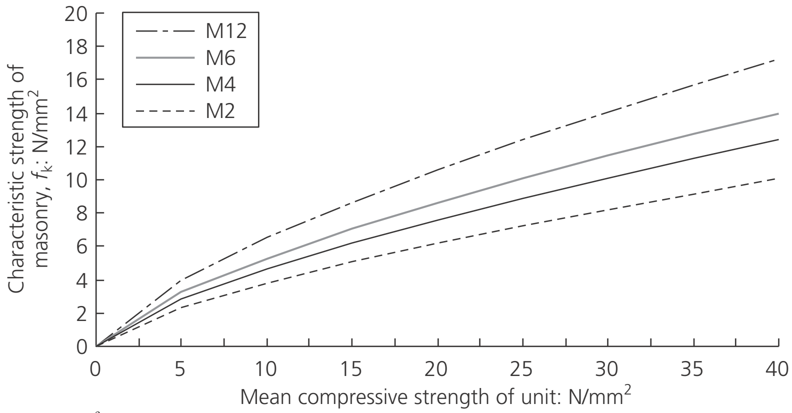
Wall without a longitudinal joint: constructed with general-purpose mortar and with standard-format aggregate concrete or autoclaved concrete blocks 215 mm high × 200 mm thick; Group 1 units. See Table A1.2(b) and Figure A1.2(b).

Table A1.2(b). Wall without a longitudinal joint: 215 mm high × 200 mm thick Group 1 blocks

$\delta = 1.18$ $K = 0.55$ (i.e. wall thickness = block thickness)
Mortar Mean compressive strength of unit (N/mm^2) to BS EN 771-3 or 4 (not normalised)

UK designation	EN 1996-1-1	2.9	3.6	5.2	7.3	10.4	17.5	22.5	30.0	40.0
(i)	M12	2.3	2.9	4.1	5.2	6.7	9.7	11.5	14.1	17.2
(ii)	M6	2.2	2.6	3.4	4.3	5.4	7.8	9.3	11.4	14.0
(iii)	M4	2.0	2.3	3.0	3.8	4.8	6.9	8.3	10.1	12.4
(iv)	M2	1.6	1.9	2.4	3.1	3.9	5.6	6.7	8.2	10.1

Figure A1.2(b). Wall without a longitudinal joint: 215 mm high × 200 mm thick Group 1 blocks



$f_k = K f_b^{\alpha} f_m^{\beta}$

f_k is the characteristic strength of masonry
K is a constant which may be taken from the National Annex
f_b is the normalised compressive strength of the units
f_m is the compressive strength of the mortar
α, β are constants which may be taken from the National Annex

f_b = CF × δ × mean compressive strength for bricks and blocks

CF is a conditioning factor

Group 1, 250 mm autoclaved concrete or concrete aggregate

Wall without a longitudinal joint: constructed with general-purpose mortar and with standard-format aggregate concrete or autoclaved concrete blocks 215 mm high × 250 mm thick; Group 1 units. See Table A1.2(c) and Figure A1.2(c).

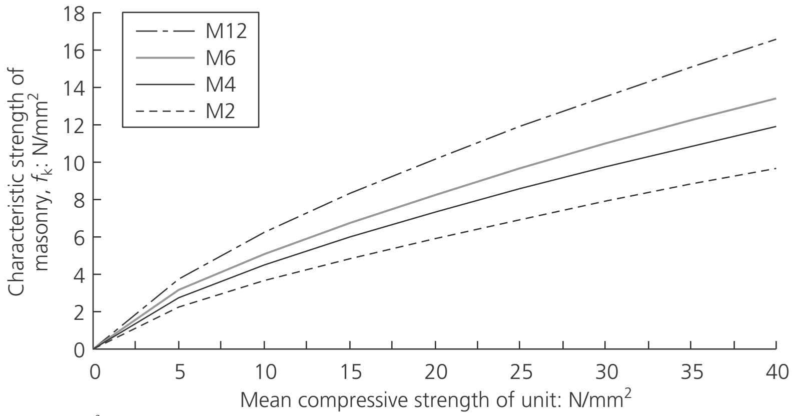
Table A1.2(c). Wall without a longitudinal joint: 215 mm high × 250 mm thick or greater Group 1 blocks

$\delta = 1.115$ $K = 0.55$ (i.e. wall thickness = block thickness)

Mortar — Mean compressive strength of unit (N/mm^2) to BS EN 771-3 or BS EN 771-4 (not normalised)

UK designation	EN 1996-1-1	2.9	3.6	5.2	7.3	10.4	17.5	22.5	30.0	40.0
(i)	M12	2.2	2.7	3.9	5.0	6.4	9.3	11.1	13.5	16.5
(ii)	M6	2.1	2.5	3.2	4.1	5.2	7.5	9.0	11.0	13.4
(iii)	M4	1.9	2.2	2.9	3.6	4.6	6.7	8.0	9.7	11.9
(iv)	M2	1.5	1.8	2.3	2.9	3.8	5.4	6.5	7.9	9.7

Figure A1.2(c). Wall without a longitudinal joint: 215 mm high × 250 mm thick or greater Group 1 blocks



$f_k = K f_b^{\alpha} f_m^{\beta}$

f_k	is the characteristic strength of masonry
K	is a constant which may be taken from the National Annex
f_b	is the normalised compressive strength of the units
f_m	is the compressive strength of the mortar
α, β	are constants which may be taken from the National Annex

f_b = CF × δ × mean compressive strength for bricks and blocks

CF is a conditioning factor

Group 2, 100 mm concrete aggregate

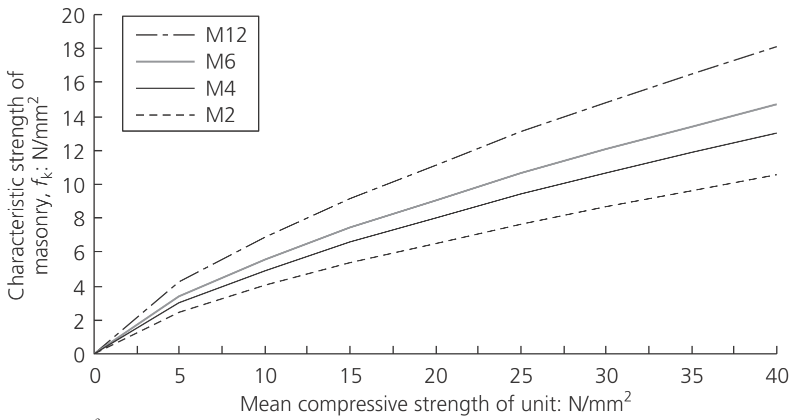
Wall without a longitudinal joint: constructed with general-purpose mortar and with standard-format aggregate concrete blocks 215 mm high × 100 mm thick; Group 2 units. See Table A1.2(d) and Figure A1.2(d).

Table A1.2(d). Wall without a longitudinal joint: 215 mm high × 100 mm thick Group 2 aggregate concrete blocks

$\delta = 1.38$ $K = 0.52$ (i.e. wall thickness = block thickness)
Mortar Mean compressive strength of unit (N/mm^2) to BS EN 771-3 (not normalised)

UK designation	EN 1996-1-1	2.9	3.6	5.2	7.3	10.4	17.5	22.5	30.0	40.0
(i)	M12	2.6	3.2	4.4	5.5	7.1	10.2	12.1	14.8	18.2
(ii)	M6	2.3	2.7	3.5	4.5	5.7	8.3	9.9	12.1	14.8
(iii)	M4	2.1	2.4	3.1	4.0	5.1	7.3	8.7	10.7	13.1
(iv)	M2	1.7	2.0	2.5	3.2	4.1	5.9	7.1	8.7	10.6

Figure A1.2(d). Wall without a longitudinal joint: 215 mm high × 100 mm thick Group 2 aggregate concrete blocks



$f_k = K f_b^{\alpha} f_m^{\beta}$

f_k is the characteristic strength of masonry
K is a constant which may be taken from the National Annex
f_b is the normalised compressive strength of the units
f_m is the compressive strength of the mortar
α, β are constants which may be taken from the National Annex

f_b = CF × δ × mean compressive strength for bricks and blocks

CF is a conditioning factor

Group 2, 200/250 mm aggregate concrete
Not applicable.

Groups 3 and 4, aggregate concrete
Group 3 and 4 aggregate concrete units have not traditionally been used in the UK, so no values are available.

Groups 2, 3 and 4, autoclaved concrete
Group 2, 3 and 4 autoclaved aerated concrete units are not used in Europe.

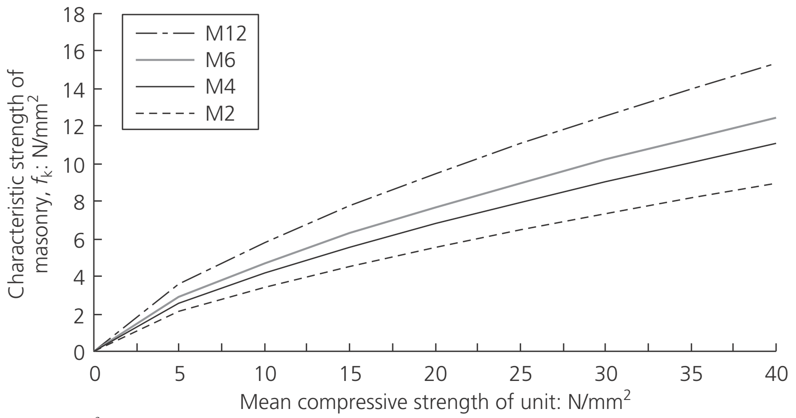
Group 1, 100 With longitudinal joints
Group 1, 100 mm autoclaved concrete or concrete aggregate
Wall with a longitudinal joint: constructed with general-purpose mortar and with standard-format aggregate concrete or autoclaved concrete blocks 215 mm high × 100 mm thick; Group 1 units. See Table A1.2(e) and Figure A1.2(e).

Table A1.2(e). Wall with a longitudinal joint: 215 mm high × 100 mm thick Group 1 blocks in 200/210 thick wall

$\delta = 1.38$ $K = 0.55$ (i.e. wall thickness = block thickness)
Mortar Mean compressive strength of unit (N/mm^2) to BS EN 771-3 or 4 (not normalised)

UK designation	EN 1996-1-1	2.9	3.6	5.2	7.3	10.4	17.5	22.5	30.0	40.0
(i)	M12	2.2	2.7	3.7	4.7	6.0	8.6	10.3	12.6	15.4
(ii)	M6	2.0	2.3	3.0	3.8	4.9	7.0	8.3	10.2	12.5
(iii)	M4	1.8	2.0	2.6	3.4	4.3	6.2	7.4	9.0	11.1
(iv)	M2	1.4	1.7	2.2	2.7	3.5	5.0	6.0	7.3	9.0

Figure A1.2(e). 215 mm high × 100 mm thick Group 1 blocks in 200/210 thick wall



$f_k = K f_b^{\alpha} f_m^{\beta}$

f_k is the characteristic strength of masonry
K is a constant which may be taken from the National Annex
f_b is the normalised compressive strength of the units
f_m is the compressive strength of the mortar
α, β are constants which may be taken from the National Annex

f_b = CF × δ × mean compressive strength for bricks and blocks

CF is a conditioning factor

Group 2, 200/250 mm autoclaved concrete or aggregate concrete

Not applicable

Group 2, 100 mm concrete aggregate

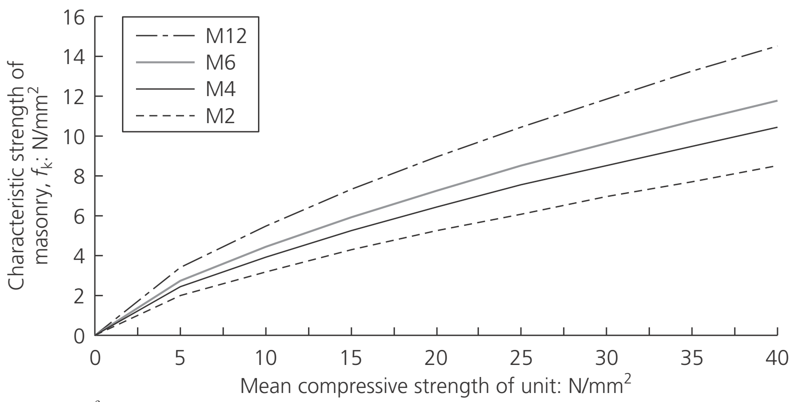
Wall with a longitudinal joint: constructed with general-purpose mortar and with standard-format aggregate concrete blocks 215 mm high × 100 mm thick; Group 2 units. See Table A1.2(f) and Figure A1.2(f).

Table A1.2(f). Wall with a longitudinal joint: 215 mm high × 100 mm thick Group 2 aggregate concrete blocks in 200/210 thick wall

$\delta = 1.38$ $K = 0.52$ (i.e. wall thickness = block thickness)
Mortar Mean compressive strength of unit (N/mm^2) to BS EN 771-3 (not normalised)

UK designation	EN 1996-1-1	2.9	3.6	5.2	7.3	10.4	17.5	22.5	30.0	40.0
(i)	M12	2.0	2.5	3.5	4.4	5.7	8.1	9.7	11.9	14.5
(ii)	M6	1.9	2.2	2.8	3.6	4.6	6.6	7.9	9.6	11.8
(iii)	M4	1.7	1.9	2.5	3.2	4.1	5.9	7.0	8.5	10.4
(iv)	M2	1.4	1.6	2.0	2.6	3.3	4.8	5.7	6.9	8.5

Figure A1.2(f). Wall with a longitudinal joint: 215 mm high × 100 mm thick Group 2 aggregate concrete blocks in 200/210 thick wall



$f_k = K f_b^{\alpha} f_m^{\beta}$

f_k is the characteristic strength of masonry
K is a constant which may be taken from the National Annex
f_b is the normalised compressive strength of the units
f_m is the compressive strength of the mortar
α, β are constants which may be taken from the National Annex

f_b = CF × δ × mean compressive strength for bricks and blocks

CF is a conditioning factor

Group 2, 200/250 mm autoclaved concrete or aggregate concrete

Not applicable

Group 1, blocks laid flat

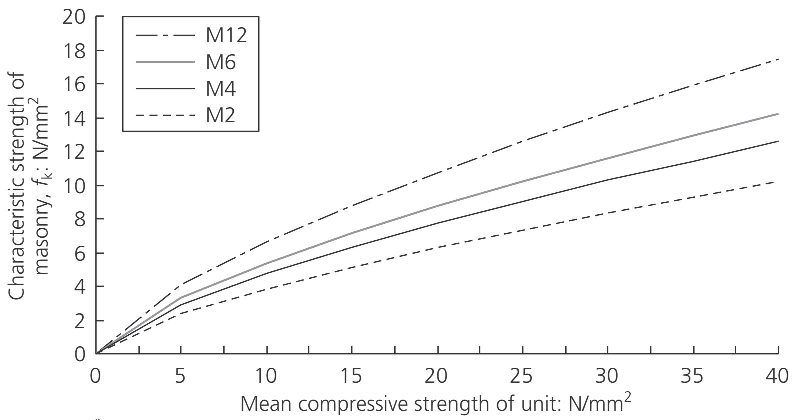
Wall without a longitudinal joint: constructed with general-purpose mortar and with standard-format aggregate concrete (with no voids) or autoclaved concrete blocks 215 mm high × 100 mm thick laid flat (normally, these units are not laid flat since a wider unit (215 mm tall and laid in the vertical direction) could readily be laid and would usually be specified); Group 1 units. Note: See Table A1.2(g) and Figure A1.2(g).

Table A1.2(g). 215 mm high × 100 mm thick Group 1 aggregate blocks laid flat

$\delta = 1.38$ $K = 0.50$ (i.e. wall thickness = block thickness)
Mortar Mean compressive strength of unit (N/mm^2) to BS EN 771-3 or 4 (not normalised)

UK designation	EN 1996-1-1	2.9	3.6	5.2	7.3	10.4	17.5	22.5	30.0	40.0
(i)	M12	2.5	3.1	4.2	5.3	6.8	9.8	11.7	14.3	17.5
(ii)	M6	2.3	2.6	3.4	4.3	5.5	8.0	9.5	11.6	14.2
(iii)	M4	2.0	2.3	3.0	3.8	4.9	7.0	8.4	10.3	12.6
(iv)	M2	1.6	1.9	2.4	3.1	4.0	5.7	6.8	8.3	10.2

Figure A1.2(g). 215 mm high × 100 mm thick Group 1 aggregate blocks laid flat



$f_k = K f_b^{\alpha} f_m^{\beta}$

f_k is the characteristic strength of masonry
K is a constant which may be taken from the National Annex
f_b is the normalised compressive strength of the units
f_m is the compressive strength of the mortar
α, β are constants which may be taken from the National Annex

f_b = CF × δ × mean compressive strength for bricks and blocks

CF is a conditioning factor

If Group 1 aggregate concrete units contain formed voids, multiply K by $(100 - n)/100$, where n is the percentage of voids which should be no more than 25%.

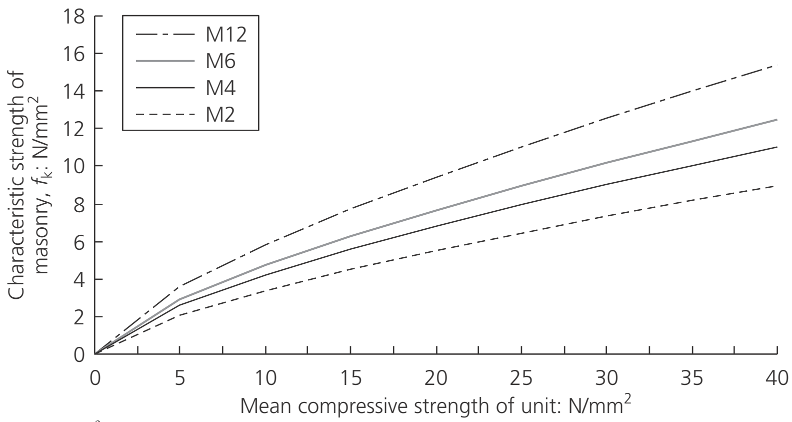
Wall with a longitudinal joint: constructed with general-purpose mortar and with standard-format aggregate concrete or autoclaved concrete blocks 215 mm high × 100 mm thick; Group 1 units. See Table A1.2(h) and Figure A1.2(h).

Table A1.2(h). Wall with a longitudinal joint: 215 mm high × 100 mm thick Group 1 blocks in 200/210 thick wall

$\delta = 1.38$ $K = 0.55$ (i.e. wall thickness = block thickness)
Mortar Mean compressive strength of unit (N/mm^2) to BS EN 771-3 or 4 (not normalised)

UK designation	EN 1996-1-1	2.9	3.6	5.2	7.3	10.4	17.5	22.5	30.0	40.0
(i)	M12	2.2	2.7	3.7	4.7	6.0	8.6	10.3	12.6	15.4
(ii)	M6	2.0	2.3	3.0	3.8	4.9	7.0	8.3	10.2	12.5
(iii)	M4	1.8	2.0	2.6	3.4	4.3	6.2	7.4	9.0	11.1
(iv)	M2	1.4	1.7	2.2	2.7	3.5	5.0	6.0	7.3	9.0

Figure A1.2(h). Wall with a longitudinal joint: 215 mm high × 100 mm thick Group 1 blocks in 200/210 thick wall



$f_k = K f_b^{\alpha} f_m^{\beta}$

f_k is the characteristic strength of masonry
K is a constant which may be taken from the National Annex
f_b is the normalised compressive strength of the units
f_m is the compressive strength of the mortar
α, β are constants which may be taken from the National Annex

f_b = CF × δ × mean compressive strength for bricks and blocks

CF is a conditioning factor

A1.3. Thin-layer mortar: bricks and blocks

Bricks

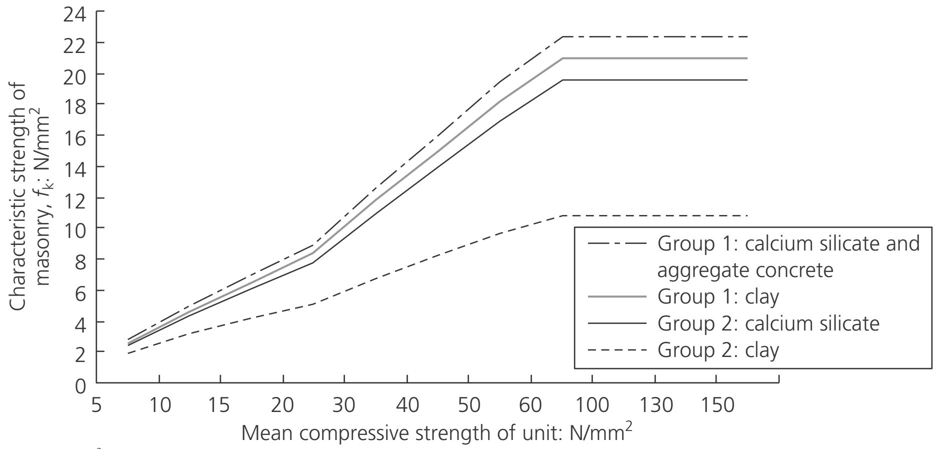
Group 1 and 2, clay, concrete and calcium silicate

Wall without a longitudinal joint: constructed with thin-layer mortar and with standard-format bricks of clay, calcium silicate and aggregate concrete. See Table A1.3(a) and Figure A1.3(a).

Table A1.3(a). Wall without a longitudinal joint: 65 mm high × 102 mm thick standard-format bricks

Unit group	Unit type	5	10	15	20	30	40	50	60	100	130	150
1	Clay brick	2.6	4.6	6.5	8.3	11.8	15.0	18.2	20.9	20.9	20.9	20.9
2	Clay brick	1.9	3.1	4.2	5.1	6.8	8.3	9.7	10.8	10.8	10.8	10.8
1	Calcium silicate	2.3	4.1	5.8	7.4	10.4	13.3	16.0	18.7	22.2	22.2	22.2
2	Calcium silicate	2.0	3.6	5.0	6.4	9.1	11.6	14.0	16.4	19.5	19.5	19.5
1	Concrete brick	2.7	4.9	7.0	8.9	12.6	16.0	19.4	22.2	22.2	22.2	22.2

Figure A1.3(a). Wall without a longitudinal joint: 65 mm high × 102 mm thick standard-format bricks



$f_k = K f_b^{\alpha} f_m^{\beta}$

f_k is the characteristic strength of masonry
K is a constant which may be taken from the National Annex
f_b is the normalised compressive strength of the units
f_m is the compressive strength of the mortar
α, β are constants which may be taken from the National Annex

f_b = CF × δ × mean compressive strength for bricks and blocks

CF is a conditioning factor

See the section on calcium silicate units at the end of this annex.

Blocks

Group 1, 100/200/250 mm autoclaved concrete and aggregate concrete

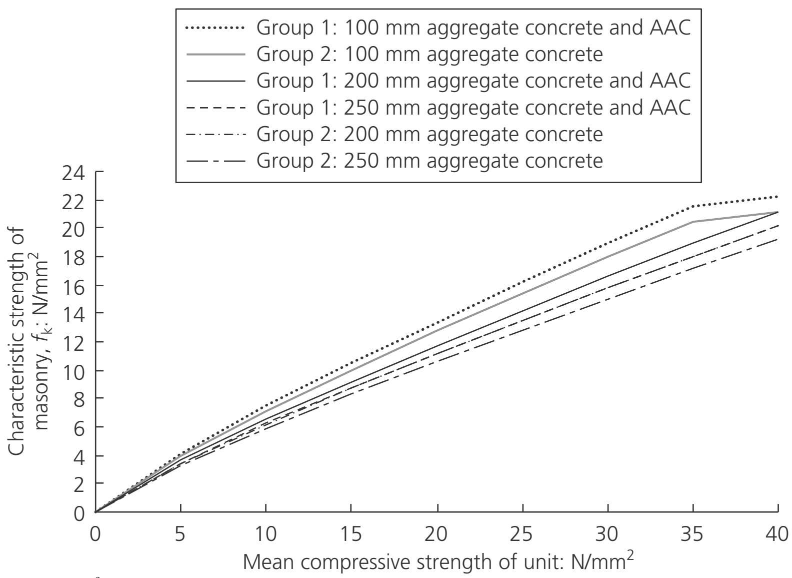
Wall without a longitudinal joint: constructed with thin-layer mortar and with standard-format blocks of aggregate concrete and autoclaved aerated concrete. See Table A1.3(b) and Figure A1.3(b).

Table A1.3(b). Wall without a longitudinal joint: 215 mm high standard-format blocks of 100, 200 and 250 mm thickness

Unit group	Unit type	Unit thickness: mm	2.9	3.6	5.2	7.3	10.4	17.5	22.5	30	35	40
1	Aggregate concrete and AAC	100	2.6	3.1	4.3	5.7	7.7	12.0	14.8	18.9	21.6	22.2
1	Aggregate concrete and AAC	200	2.3	2.7	3.7	5.0	6.7	10.5	13.0	16.6	18.9	21.2
1	Aggregate concrete and AAC	250	2.2	2.6	3.6	4.8	6.4	10.0	12.4	15.8	18.0	20.2
2	Aggregate concrete	100	2.5	3.0	4.1	5.4	7.3	11.4	14.1	18.0	20.5	21.1
2	Aggregate concrete	200	2.2	2.6	3.6	4.7	6.4	10.0	12.3	15.8	18.0	20.1
2	Aggregate concrete	250	2.1	2.5	3.4	4.5	6.1	9.5	11.8	15.0	17.1	19.2

AAC: autoclaved aerated concrete.

Figure A1.3(b). Wall without a longitudinal joint: 215 mm high standard-format blocks of 100, 200 and 250 mm thickness



$f_k = K f_b^{\alpha} f_m^{\beta}$

f_k	is the characteristic strength of masonry
K	is a constant which may be taken from the National Annex
f_b	is the normalised compressive strength of the units
f_m	is the compressive strength of the mortar
α, β	are constants which may be taken from the National Annex

f_b = CF × δ × mean compressive strength for bricks and blocks

CF is a conditioning factor

Calcium silicate bricks

The conditioning factor for calcium silicate bricks is 0.8 (see Annex D of EN 771-2:2003). The values shown in the tables in this annex are based on this value to calculate f_k. However, care is needed. The reader should be aware that, in the UK, calcium brick production is now limited to very few companies. When considering the compressive strength of calcium silicate brickwork, there is a tendency for the industry to quote ‘a minimum of Class XX for compressive strength’ in its literature. This too is covered in Annex D of EN 771-2. A Class 30 unit, for example, usually refers to 30 N/mm^2 – not for the *mean compressive strength* (as one might imagine) but for the *normalised compressive strength*. So, if a Class 30 calcium brick is specified, $f_b = 30\ N/mm^2$.

There is the possibility that some manufacturers may do one thing while other manufacturers do another: if in doubt, always check with the manufacturer.

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