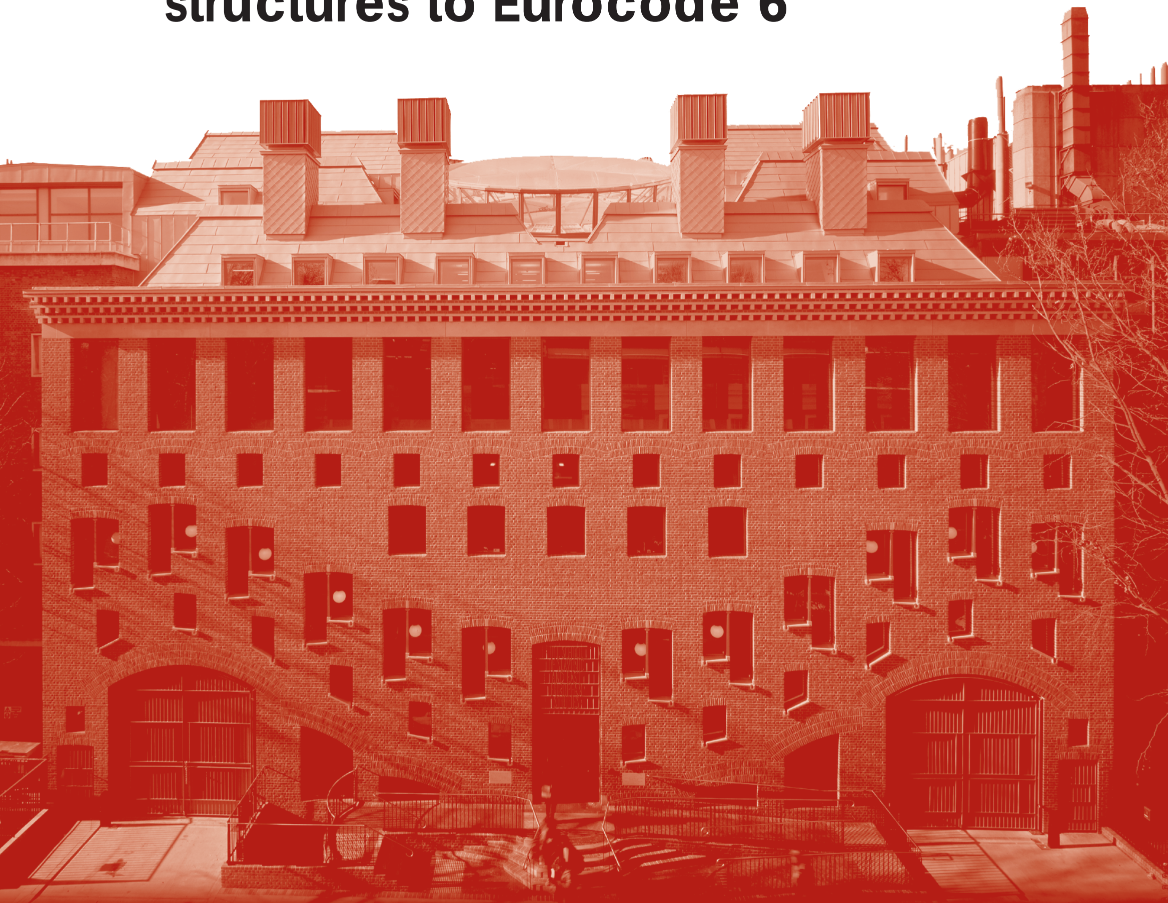
The Institution of Structural Engineers

February 2008

Manual for the design of plain masonry in building structures to Eurocode 6



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Manual for the design of plain masonry in building structures to Eurocode 6

February 2008

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Notation

Latin upper case letters

A	Loaded horizontal gross cross-sectional area of a wall
A_b	Loaded area
A_{ef}	Effective area of bearing
E	Short term secant modulus of elasticity of masonry
E_i	Modulus of elasticity of member, i
$E_{longterm}$	Long term modulus of elasticity of masonry
F_d	Design compressive or tensile resistance of a wall tie
F_t	Design horizontal tie force
G_k	Characteristic value of a permanent action
$G_{k,inf}$	Lower characteristic value of a permanent action
$G_{k,sup}$	Upper characteristic value of a permanent action
I	Second moment of area
I_i	Second moment of area of member, i
K	Constant used in the calculation of the compressive strength of masonry
L_a	The lesser of the distance between loadbearing members in the direction of a tie or five times the clear height of the wall
M_{Ed}	Design value of the moment applied
M_{Edu}	Design value of the moment above a floor
M_{Edf}	Design value of the moment below a floor
M_i	End moment at node, i
M_{id}	Design value of the bending moment at the top or the bottom of the wall
M_{md}	Design value of the greatest moment at the middle of the height of the wall
M_{Rd}	Design value of the moment of resistance
M_{Rds}	Design value of the moment of resistance at the base of a wall due to gravity action
N	Sum of the design vertical actions on a building
N_{ad}	The maximum design arch thrust per unit length of wall
N_{Ed}	Design value of the vertical load
N_{Edc}	Design value of a concentrated vertical load
N_{Edf}	Design value of the load out of (applied by) a floor
N_{Edu}	Design value of the load above the floor
N_{id}	Design value of the vertical load at the top or bottom of a wall or column

N_{md}	Design value of the vertical load at the mid-height of a wall or column
N_{Rd}	Design value of the vertical resistance of a masonry wall or column
N_{Rdc}	Design value of the vertical concentrated load resistance of a wall
N_s	Number of storeys including ground and basement
Q_k	Characteristic value of a single variable action
$Q_{k,1}$	Characteristic value of the leading variable action 1
$Q_{k,i}$	Characteristic value of the accompanying variable action *i*
V_{Ed}	Design value of a shear load
V_{Rd}	Design value of the shear resistance
W_c	Width of compressive stress block under design dead load
W_{Ed}	Design lateral load per unit area
Z	Elastic section modulus of a unit height or length of the wall

Latin lower case letters

a, a_1	Distance from the end of a wall to the nearest edge of a loaded area
b_{ci}	Width of stressed area
d_a	Deflection of an arch under the design lateral load
e_{he}	Eccentricity at the top or bottom of a wall, resulting from horizontal loads
e_{hm}	Eccentricity at the middle of a wall, resulting from horizontal loads
e_i	Eccentricity at the top or the bottom of a wall
e_{init}	Initial eccentricity
e_k	Eccentricity due to creep
e_m	Eccentricity due to loads
e_{mk}	Eccentricity at the middle of the wall
f_b	Normalised mean compressive strength of a masonry unit
f_d	Design compressive strength of masonry in the direction being considered
f_k	Characteristic compressive strength of masonry
f_m	Compressive strength of masonry mortar
f_{vd}	Design shear strength of masonry
f_{vk}	Characteristic shear strength of masonry
f_{vk0}	Characteristic initial shear strength of masonry, under zero compressive stress
f_{xk1}	Characteristic flexural strength of masonry having a plane of failure parallel to the bed joints
f_{xk2}	Characteristic flexural strength of masonry having a plane of failure perpendicular to the bed joints
h	Clear height of a masonry wall
h_a	Clear height of a masonry wall between restraining surfaces

h_c	Height of a wall to the level of the load
h_{ef}	Effective height of a wall
h_i	Clear height of masonry wall, *i*
h_{tot}	Total height of a structure, from the top of the foundation, or a wall, or a core
k_{tef}	Factor used to obtain effective thickness of a cavity wall
l	Length of a wall (between other walls, between a wall and an opening, or between openings)
l_a	The length or the height of the wall between supports capable of resisting an arch thrust
l_c	Length of the compressed part of a wall
l_{ch}	Length of chase
l_{efm}	Effective length of a bearing at mid height of a wall
l_i	Clear span of member, *i*
l_m	Length of masonry wall between movement joints
n	Number of storeys
n_i	Stiffness factor of members, *i*
n_t	Number of wall ties or connectors per m^2 of wall
$q_{lat,d}$	Design lateral strength per unit area of wall
r	Arch rise
t	Thickness of a wall
$t_{ch,h}$	Maximum depth of a horizontal or inclined chase
$t_{ch,v}$	Maximum depth of a vertical chase or recess without calculation
t_{ef}	Effective thickness of a wall
t_i	Thickness of wall *i*
t_{min}	Minimum thickness of a wall
t_r	Thickness of wall behind chase or recess
u	Value used to obtain the reduction factor in the middle height of the wall
w_c	Width of chase
w_i	Uniformly distributed design load, *i*
x	Maximum dimension of opening
z	Lever arm

Greek letters

α	Power used for obtaining characteristic compressive strength of masonry
α_t	Coefficient of thermal expansion of masonry
α_2	Bending moment coefficient
β	Enhancement factor for concentrated loads (Section 5.3.1.3 only)
β	Power used for obtaining characteristic compressive strength of masonry (Section 4.3.2 only)
γ_G	Partial factor for permanent actions, also accounting for model uncertainties and dimensional variations
$\gamma_{G,inf}$	Partial factor for permanent actions in calculating lower design value
$\gamma_{G,sup}$	Partial factor for permanent actions in calculating upper design value
γ_M	Partial factor for a material property, also accounting for model uncertainties and dimensional variations
γ_Q	Partial factor for variable actions, also accounting for model uncertainty and dimensional variations
$\varepsilon_{c\infty}$	Final creep strain
ε_{el}	Elastic strain of masonry
η	Factor for use in calculating the out-of-plane eccentricity of loading on walls
λ	Value used to obtain reduction factor within height of the wall
λ_c	Value of the slenderness ratio up to which eccentricities due to creep can be neglected
μ	Orthogonal ratio of the flexural strengths of masonry
ρ_n	Reduction factor
ρ_t	Stiffness coefficient
σ_d	Design compressive stress (under permanent actions only for laterally loaded walls)
υ	Angle of inclination to the vertical of the structure
ϕ_∞	Final creep coefficient of masonry
Φ	Reduction factor
Φ_i	Reduction factor at the top or bottom of the wall
Φ_m	Reduction factor within the middle height of the wall
ψ_0	Factor for combination value of a variable action
ψ_1	Factor for frequent value of a variable action
ψ_2	Factor for quasi-permanent value of a variable action

Foreword

The Eurocodes for the Design of Plain Masonry Structures (Eurocode 6) comprising BS EN 1996-1-1: 2005, BS EN 1996-1-2: 2005 and BS EN 1996-2: 2006 are available from the British Standards Institution. The UK National Annexes setting out the Nationally Determined Parameters (NDPs) to be used in the UK are also available from the British Standards Institution. They provide the necessary information to enable BS EN 1996 designs to comply with the Building Regulations. BS EN 1996-3: 2006 is not included in this *Manual* as the simplified calculation methods it contains are considered to be of limited use in the UK, although relevant parts of the scope are covered.

These documents, together with previously published documents BS EN 1990: 2002: *Eurocode - Basis of Structural Design* and BS EN 1991: 2002: *Eurocode 1 - Actions on Structures*, and their respective National Annexes, provide a suite of information for the design of most types of unreinforced masonry building structures in the UK. After a period of co-existence, the current National Standards will be withdrawn and replaced by the Eurocodes.

This *Manual* follows the basic format of the *Manual for the design of plain masonry in building structures* first published by the Institution of Structural Engineers in 1997 and revised in 2005. It provides guidance on the design of loadbearing masonry and masonry infill to structural frames. The limit state design approach follows on from that used for many years in BS 5628. The *Manual* covers structural fire design of masonry and selection of materials and execution of masonry. Established good practice not covered in the Eurocode is included, suitably identified. Expressions incorporating UK National Annex values are distinguished from the corresponding general expressions in Eurocode 6, and the values of NDPs adopted by the UK are shown in bold.

The preparation of this *Manual* was partly funded by The Aircrete Products Association, The Brick Development Association and The Concrete Block Association. This funding allowed the appointment of Neil Tutt of Jenkins and Potter to act as a Consultant to the Task Group with the role of researching and writing the initial drafts for consideration by the Task Group and of preparing the final text.

Special thanks are due to all of the members of the Task Group and to their organisations, who have given their time voluntarily. In conjunction with Neil Tutt they ensured that the original programme was achieved. I am also very grateful to Berenice Chan for acting as secretary to the Task Group: she fulfilled this considerable task with tolerance and skill, meticulously recording our decisions. During the review process, members of the Institution provided invaluable comment on the draft *Manual* that has contributed to its improvement.

I join with all of the other members of the Task Group in commending this *Manual* to the industry.

Dr John Moore
Chairman

1 Introduction

1.1 Aims of the *Manual*

The *Manual* provides guidance on the design of plain masonry in building structures in accordance with BS EN 1996-1-1[1], BS EN 1996-1-2[2] and BS EN 1996-2[3] (Parts of the Eurocode 6 suite of masonry design codes) and the information contained in the National Annexes[4-6] to these parts. Plain masonry is an assemblage of structural units, either laid in-situ or constructed in prefabricated panels, in which the structural units are bonded and solidly put together with mortar or grout, but without structural reinforcement. For simplicity, Eurocode 6 will be referred to as EC6 in the *Manual*. The *Manual* is intended to encourage the use of structural masonry in the United Kingdom and in consequence makes reference to UK practice accordingly. The guidance principles are, however, intended to be applicable worldwide. It must be recognised that local, regional and national variations to design requirements should be allowed for as necessary when using the *Manual* outside the UK. The *Manual* is one of a series dealing with the principal structural Eurocodes.

1.2 Eurocode system

The structural Eurocodes were initiated by the European Commission but are now produced by the Comité Européen de Normalisation (CEN) which is the European standards organisation, its members being the national standards bodies of the EU and EFTA countries, e.g. BSI.

In the UK, BSI is publishing the design standards as full European Standards EN:

BS EN 1990: Eurocode: Basis of structural design (referred to as EC0 in the *Manual*)

BS EN 1991: Eurocode 1: Actions on structures (EC1)

- Part 1-1: General actions – Densities, self-weight and imposed loads
- Part 1-2: General actions on structures exposed to fire
- Part 1-3: General actions – Snow loads
- Part 1-4: General actions – Wind loads
- Part 1-5: General actions – Thermal actions
- Part 1-6: Actions during execution
- Part 1-7: Accidental actions from impact and explosions
- Part 2: Traffic loads on bridges
- Part 3: Actions induced by cranes and machinery
- Part 4: Actions in silos and tanks

BS EN 1992: Eurocode 2: Design of concrete structures (EC2)

BS EN 1993: Eurocode 3: Design of steel structures (EC3)

BS EN 1994: Eurocode 4: Design of composite steel and concrete structures (EC4)

BS EN 1995: Eurocode 5: Design of timber structures (EC5)

BS EN 1996: Eurocode 6: Design of masonry structures (EC6)

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

BS EN 1997: Eurocode 7: Geotechnical design (EC7)
BS EN 1998: Eurocode 8: Earthquake resistant design of structures (EC8)
BS EN 1999: Eurocode 9: Design of aluminium alloy structures (EC9)

All Eurocodes follow a common editorial style. The codes contain 'Principles' and 'Application rules'. Principles are identified by the letter P following the paragraph number. Principles are general statements and definitions for which there is no alternative, or requirements and analytical models for which no alternative is permitted unless specifically stated.

Application rules are generally recognised rules which comply with the Principles and satisfy their requirements. Alternative rules may be used provided that compliance with the Principles can be demonstrated, however the resulting design cannot be claimed to be wholly in accordance with the Eurocode although it will remain in accordance with Principles.

Each Eurocode contains some parameters that are left open for national choice, known as Nationally Determined Parameters (NDPs). Recommendations for each NDP are given in informative Notes. National decisions on the choices are given in a National Annex for each member state. Within this *Manual* the values of NDPs adopted by the UK are shown in **bold**. This system has been adopted to warn the reader that a different value may apply if the design is to be constructed outside the UK.

1.3 Scope of the *Manual*

The range of structures covered by the *Manual* is limited to building structures that do not rely on bending in masonry for their overall stability (e.g. sway frame buildings). However, the design of individual masonry elements subject to lateral loading and involving bending for their resistance is included. The structural design of reinforced and prestressed masonry is specifically excluded from the *Manual*, as are retaining walls and arched structures. The exception to this is the use of bed joint reinforcement in laterally loaded wall panels and for crack control. The design of both loadbearing masonry and masonry infill panels to framed structures in accordance with EC6[1-3] is covered by the *Manual*. EC6 deals with the requirements for resistance, serviceability and durability of structures. Part 1-1 of EC6[1] is specifically not valid for masonry with a plan area of less than $0.04m^2$.

As referred to in Section 1.1, this *Manual* deals with the unreinforced Section of Part 1-1[1], with Part 1-2[2] and with Part 2[3]. It also includes guidance on some matters included in BS 5628 Parts 1[7] and 3[8] which are not dealt with in EC6, which relate to good practice and workmanship. It does not deal with EC6 Part 3[9]. Part 3 provides simplified calculation methods for walls subjected to various types of loading. The methods are consistent with the rules given in EC6 Part 1[1], but are more conservative with respect to conditions and limitations of use. However, as they differ from the simple rules in Approved Document A[10] to the Building Regulations[11], they will be of limited use in relation to UK practice, although the section on 'walls subjected to limited lateral load but no vertical loads' (internal partition walls) is similar to the guidance given in Section 6.10 of this *Manual*. References to Building Regulations in this *Manual* are those for England and Wales. However, those applicable to Scotland or Northern Ireland should be substituted if the structure will be constructed there.

1.4 Contents of the *Manual*

The *Manual* covers the following:

- choice of structural form (conceptual design)
- choice of materials
- general principles of limit state design for masonry walls and columns
- design of loadbearing masonry
- design of laterally loaded masonry
- details and construction
- design for fire.

1.5 National Annex

The scope of the UK National Annex[4-6] for each part of EC6[1-3] is defined in the Foreword of the respective parts.

Thus, the UK National Annex[4] to BS EN 1996-1-1 is essential to the use of the Code in the UK. It provides information on:

- values and/or classes where alternatives are given in BS EN 1996-1-1[1]
- values to be used where a symbol only is given in BS EN 1996-1-1[1]
- country specific data
- the procedure to be used when BS EN 1996-1-1[1] gives alternatives
- decisions on the application of Informative Annexes
- references to non-contradictory complementary information.

National choice is allowed in BS EN 1996-1-1[1] in the clauses below.

- 2.4.3(1)P Ultimate limit states
- 2.4.4(1) Serviceability limit states
- 3.2.2(1) Specification of masonry mortar
- 3.6.1.2(1) Characteristic compressive strength of masonry other than shell bedded
- 3.6.2(3), (4) & (6) Characteristic shear strength of masonry
- 3.6.3(3) Characteristic flexural strength of masonry
- 3.7.2(2) Modulus of elasticity
- 3.7.4(2) Creep, moisture expansion or shrinkage and thermal expansion
- 4.3.3(3) & (4) Reinforcing steel
- 5.5.1.3(3) Effective thickness of masonry walls
- 6.1.2.2(2) Slenderness ratio λ_c below which creep may be ignored
- 8.1.2(2) Minimum thickness of walls
- 8.5.2.2(2) Cavity walls
- 8.5.2.3(2) Double-leaf walls
- 8.6.2(1) Vertical chases and recesses
- 8.6.3(1) Horizontal and inclined chases.

The National Annex to BS EN 1996-1-2[5] gives information on national choice in relation to the clauses below.

- 2.2(2) Actions
- 2.3(2) Design values of material properties
- 2.4.2(3) Member analysis
- 3.3.3.1(1) Thermal elongation
- 3.3.3.2(1) Specific heat capacity
- 3.3.3.3 Thermal conductivity
- 4.5(3) Assessment by tabulated data
- Annex B Values of t_F and l_F.

The National Annex to BS EN 1996-2[6] gives information on national choice in relation to the clauses below.

- 2.3.4.2(2) Spacing of movement joints
- 3.5.3.1(1) Pointing.

The UK National Annexes are published by BSI. They are informative and provide the necessary information to enable BS EN 1996[1-3] designs to comply with the Building Regulations[11]. Individual countries are not permitted to publish National versions of the Eurocodes with the information contained in the National Annexes incorporated into the text. For that reason, information taken from the National Annexes applicable to the UK is identified as such in this publication with UK NDPs appearing in **bold type.**

1.6 Use of the *Manual*

The *Manual* is for use by structural engineers in the preparation of their designs in accordance with EC6[1-6] . It is intended that the *Manual* should provide the information necessary to carry out design calculations. Figure 1.1 shows alternative routes for masonry design.

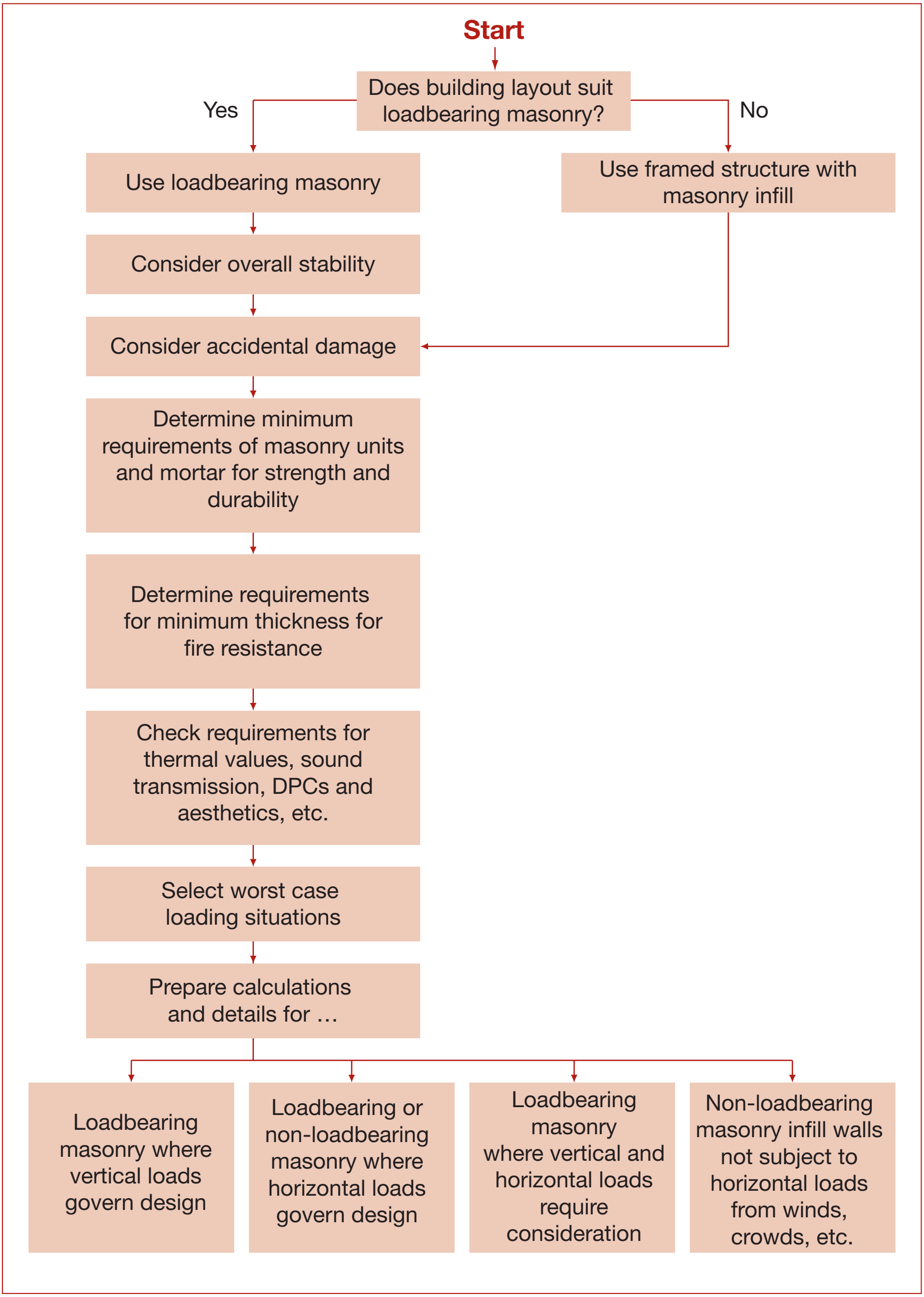


Fig 1.1 Alternative routes for masonry design

2.1 General

The designer responsible for the overall stability of the structure should also be responsible for the compatibility of the design and details of parts and components. There should be no doubt of this responsibility for overall stability when some or all of the design and details are not made by the same designer.

The designer should take account of their responsibilities under the Construction (Design and Management) Regulations[12].

Appendix A to the *Manual* contains a list of design data, which it is suggested should be considered at the inception of any design.

The structure should be so arranged that it can transmit dead, imposed and wind loads in a direct manner to the foundations. The general arrangement should lead to a robust and stable structure that will not collapse progressively and/or to a degree that is disproportionate to the cause, under the effects of misuse or accidental damage to any one element.

The arrangement and orientation of masonry walling is very significant for the stability and robustness of the overall structure. The layout can also have a significant influence on the behaviour of individual elements, particularly with respect to accidental damage.

Building forms that consist predominantly of isolated loadbearing masonry walls, piers and columns require careful consideration, since they have few alternative load paths. Buildings of this form are not often used, and this *Manual* does not offer guidance on their design.

A good design will take account of dimensional coordination of the masonry so as to minimise the cutting of units and making up of levels.

The subsections that follow give examples of efficient structural plan forms if loadbearing masonry is to be used. Other layouts may require an independent structural frame for stability.

2.2 Stability

Lateral stability in two orthogonal directions should be provided by a system of strongpoints within the structure so as to produce a ‘braced’ structure, i.e. one in which the walls will not be subject to additional eccentricities arising from sway. Strongpoints can generally be provided by orientating the walls uniformly about the two horizontal axes of the structure and in some cases may be provided by the walls enclosing stairs, lift shafts or service ducts. It is preferable for the strongpoints to be distributed throughout the structure and arranged so that their combined shear centre is located approximately on the line of the resultant in plan of the applied overturning forces (see Figure 2.1). Where this is not possible, the resultant additional torsional moments must be considered when calculating the load carried by each strongpoint (see Figure 2.2).

Strongpoints should be effective throughout the full height of the building, although they may be reduced in the upper storeys. If it is necessary for the strongpoints to be discontinuous at one level, provision needs to be made to transfer the forces to other strongpoints.

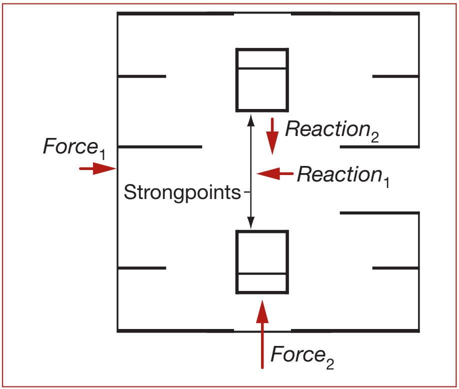


Fig 2.1 Symmetrical plan strongpoints

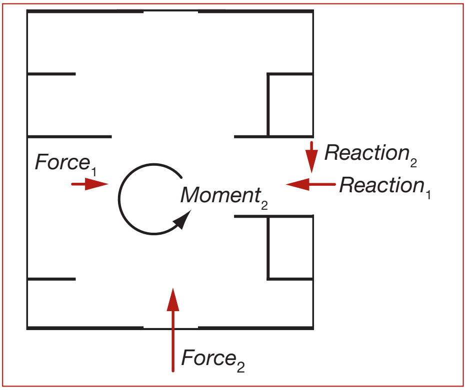


Fig 2.2 Asymmetrical plan strongpoints

2.3 Cellular plan form

The cellular plan form consists of a number of loadbearing walls parallel to the horizontal axes of the building and usually intersecting to produce a cellular compartmented layout (see Figure 2.3). Generally the cellular plan form produces the most stable and robust structure.

2.4 Crosswall construction

The arrangement of the loadbearing walls in crosswall construction is an array of parallel walls, usually at right-angles to the longitudinal axis of the building. The combined strength of both strongpoint and crosswall construction relies on the floors acting as horizontal diaphragms (see Figure 2.4).

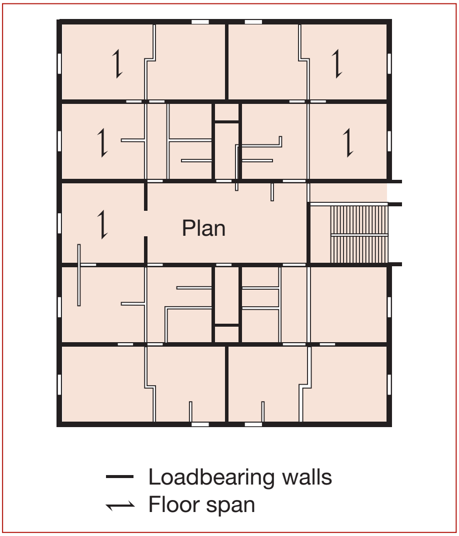


Fig 2.3 Cellular wall plan

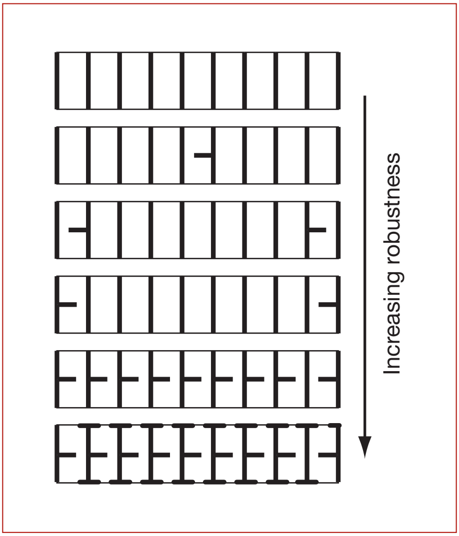


Fig 2.4 Crosswall plan

2.5 Spine-wall construction

Crosswall construction is appropriate where repetitive floor plans allow the loadbearing crosswalls to line through on all floors and hence to carry their loading directly down to the foundation. Where such a repetitive floor plan is not appropriate, spine-wall construction may provide a suitable alternative particularly where, for overall stability, strongpoints are provided by staircases, lift shafts and end walls. The combined strength of strongpoints and spine-walls relies on the floors acting as horizontal diaphragms (see Figure 2.5).

2.6 Geometric sections

Geometric profiles can be readily formed in masonry and are particularly suitable for use in tall and slender single-storey, open plan structures such as assembly halls, theatres, churches and warehouses. The scope for such sections is wide and includes diaphragm walls, fin walls, channel sections, chevron walls, etc. The principle employed is to create bending stiffness in the plan shape of the wall elements (see Figure 2.6). The section should be sized to suit masonry unit dimensions to avoid excessive cutting and poor bonding.

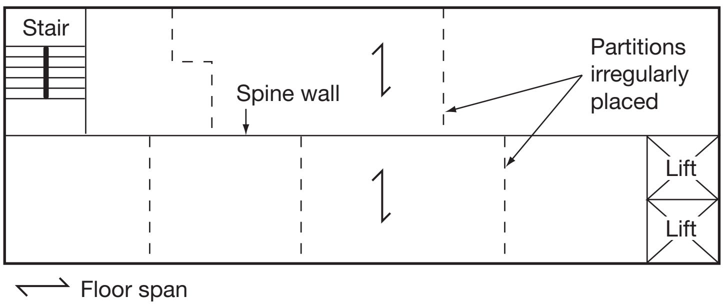


Fig 2.5 Spine-wall plan

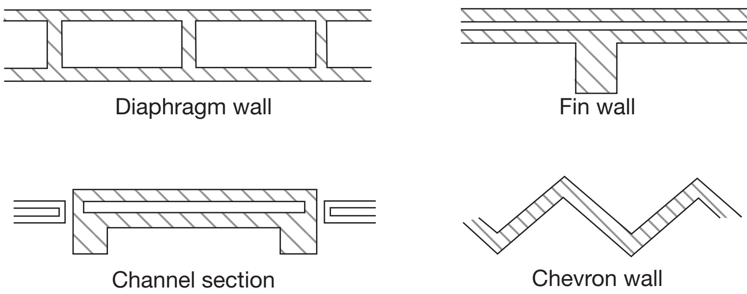


Fig 2.6 Typical geometric wall sections

2.7 Structural robustness

A well designed and well detailed structure with an appropriate choice of materials will normally satisfy the requirements of structural robustness.

Elements whose failure would cause collapse of more than a limited part of the structure adjacent to them must be avoided. Where this is not possible, alternative load paths should be provided or the element in question strengthened. These are designated as key elements. The adequacy of the junctions or connections between the masonry walls and floors and roofs is an important consideration. In certain circumstances (e.g. buildings with precast concrete or timber floors), all members of the structure should be effectively tied together in both the longitudinal and transverse directions (see Section 5.3.1).

Careful consideration may be necessary over the location and dimensions of openings as well as the position of any movement joints so that the integrity of the structure is not impaired.

In its simplest form, crosswall construction has stability only about the minor axis. In the longitudinal direction, bracing to the structure as a whole should be provided, for example, by spine or perimeter walls, by buttressing or by strongpoints, such as stair or lift shafts, located at each end of or midway along the building (see Figure 2.5).

In the case of both spine-wall and crosswall construction the location of primary joints through the structure and of secondary masonry elemental movement joints must be carefully considered in the assessment of stability and robustness.

2.8 Movement joints

Joints should be provided to minimise the effects of movement caused by drying shrinkage, other moisture movements, temperature variations, creep and settlement in masonry elements and between masonry elements and other parts of the structure.

The effectiveness of movement joints depends on their location. In masonry construction, there are two distinct types of movement joint: primary movement joints that should divide the structure into individual and independently stable parts and secondary elemental movement joints that divide the masonry construction into individual portions. The structure and elements on each side of the joint should be independently stable and robust.

Where movement joints are used, it is essential to provide a continuous joint through any finishes (e.g. plaster), features (e.g. cappings and copings), attached cladding and similar elements. It should be noted that the need for secondary movement joints may be able to be reduced by calculation of the likely movement or by use of bed joint reinforcement.

Primary movement joints are used to reduce the influence of overall dimensional changes or distortions of the total structure, and are usually positioned at changes in direction, significant changes in dimension of plan or height, or changes in the form of construction either of the structure or of its foundations. In long uniform structures these joints would normally be provided at 40 to 50m centres and be at least 25mm in width.

Primary movement joints should pass through the whole of the structure above ground level (see Figure 2.7) and be in one plane. Consideration should be given to the need to carry the joint through the foundations.

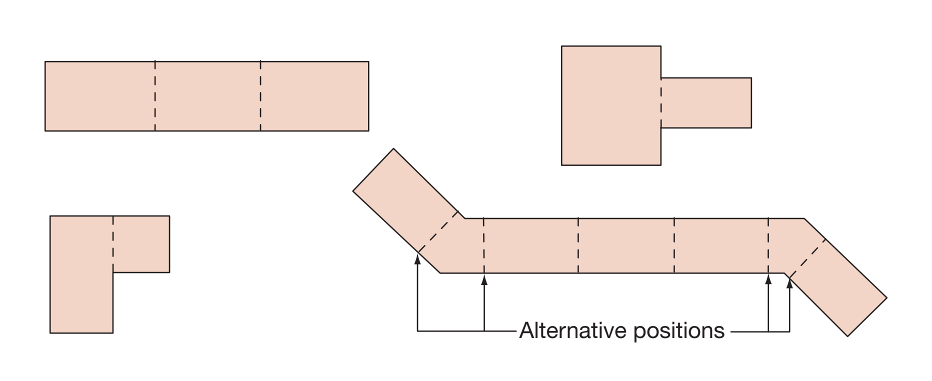


Fig 2.7 Suggested location of primary movement joints

The purpose of secondary elemental movement joints is usually to accommodate movements arising from material behaviour; these may be long term, short term or differential movements including local structural distortions.

Clay masonry generally exhibits long-term moisture expansion, whereas concrete and calcium silicate masonry units experience drying shrinkage. Movement joints should be provided accordingly. In long walls, concrete and calcium silicate masonry may require the inclusion of expansion joints. It may be appropriate to include debonding ties across movement joints where it is desirable to incorporate some degree of restraint or compatibility of deflection.

The spacing and location of secondary elemental movement joints need to be carefully considered. Features of the building that should be considered when determining the joint positions are as follows:

- intersections of walls, piers, floors, etc.
- internal and external corners
- short return walls
- window and door openings
- change in height or thickness of the wall
- chases in the wall
- beam seatings or other elements imposing concentrated loads
- areas of cantilevered construction
- areas of arched construction
- parapets
- changes in material.

Guidance on the spacing of these joints is given in the National Annex[6] to EC6 Part 2, see Table 2.1. Spacing of these joints may also be obtained by calculation, making the appropriate allowances for material properties. Care should be taken to ensure that the positioning of these joints does not compromise the structural behaviour of the masonry, e.g. where arching action is involved. Bed joint reinforcement may also be used for controlling movement in masonry walls, see Note **a** to Table 2.1.

Table 2.1 Recommended maximum horizontal distance, l_m, in m, between vertical movement joints for external unreinforced, non-loadbearing walls[a]

Type of masonry	Restrained at the top and bottom of the wall
Clay masonry	**15**
Calcium silicate masonry	**9**[b]
Aggregate concrete and manufactured stone masonry	**9**[b]
Autoclaved aerated concrete masonry	**9**[b]
Natural stone masonry	**20**[c]

Notes

a The maximum horizontal spacing of vertical movement joints may be increased for walls containing bed joint reinforcement subject to expert advice.

b This value applies when the ratio of length to height of panel is 3 to 1 or less. It should be reduced for masonry walls having a length to height ratio greater than 3.

c When using this figure, movement joints should be located at not more than 8m from the corner.

d This table is based on data from the National Annex to EC6 Part 2[6].

The width of the movement joints, both horizontal and vertical, should take account of the range of anticipated movements and whether the movements are permanent or reversible. In addition, the joint width should take account of the compressibility or elasticity of the joint filler and any sealant. In certain circumstances, particularly any joints in internal masonry walls, the fire resistance of the filler and sealant may need to be considered.

Typical vertical movement joint widths in clay masonry are about 15 to 20mm wide and those in calcium silicate masonry (where there is some permanent shrinkage) are in the range 12 to 18mm. Concrete block masonry (which can exhibit an even higher shrinkage) can have movement joint widths ranging from simple butt joints, for internal blockwork, up to 20mm for south and west facing external walls.

In the case of horizontal movement joints the necessary joints are not so dependent on the material from which the masonry is built, but on the cause of the movement. For simple thermal and moisture movements, which are predominantly reversible, the joint widths may be as little as 10 to 15mm, but where movements are governed by long-term deflections and creep then joint widths of up to 100mm or so have been found to be necessary if the masonry is not to be subjected to any imposed loads.

The uninterrupted height and length of the outer leaf of external cavity walls should be limited so as to avoid undue loosening of the ties arising from differential movements between the two leaves. It is considered to be good practice in the UK that the outer leaf should be supported at intervals of not more than every third storey or 9m, whichever is less. However, for buildings not exceeding four storeys or 12m in height, whichever is less, the outer leaf may be uninterrupted for its full height. Alternatively, calculations may be carried out. The effects of horizontal movement should also be considered. Further guidance may be obtained from CIRIA Technical Note 107[13] and BDA Design Note 10[14] . Information may also be found in BRE Digests 227[15] , 228[16] and 229[17].

2.9 Interaction with other parts of the structure

This section refers to the relative behaviour of the masonry elements with other parts of the structure, particularly at interfaces or junctions or where composite action is required. Compared with most other materials used in the structure of a building, unreinforced masonry is relatively stiff and brittle. It does not readily absorb distortions arising from movement or displacements, nor readily redistribute high localised stresses. Some examples requiring consideration are:

- masonry panels on suspended beams or slabs that may crack because of the deflection of the supporting structure
- diaphragm action of floors transmitting lateral forces to strongpoints or shear walls
- lateral restraint to walls by floors
- infill masonry panels (which should be individually supported and connected to the surrounding frame, whilst allowing for relative movement)
- uplift and suction arising from wind (special attention needed at roof/wall junctions)
- shrinkage of in-situ concrete where supporting, or supported by, masonry elements.

Particularly in cases of precast concrete floor units and timber floor joists and roof trusses (see Figure 2.8) the designer must be satisfied that these elements can act as horizontal diaphragms where so assumed and that the connections to the masonry supporting structure can transmit the forces resulting from the interaction.

Lateral deflections of a reinforced concrete or steel frame may induce cracking of infill cladding. Frame shortening may impose load on infill masonry unless a horizontal compression joint is provided, see Section 2.10.

2.10 Infill masonry to framed structures

Masonry infilling should not generally be used to provide the bracing to framed structures. Where masonry infilling is used for this purpose, the walls are not usually required to carry gravity loads from the structure but are subjected to in-plane loads. Where infill also provides the cladding to the building it will also need to resist wind loads normal to the wall. Due consideration must be given to making sure that bracing walls are identified as such, and to the effects of possible removal of these walls at a later date.

Infill masonry panels, when used as bracing, should be fixed tightly to the surrounding structural frame for the efficient bracing of the structure. Regard should be paid to the possible shrinkage of calcium silicate and concrete masonry panels making the pinning ineffective. Movement joints within the panel, either primary or secondary, should be avoided. Similarly, openings that might impair the ability of the panel to brace the structure should be carefully examined. Load sharing arising from secondary effects (e.g. frame shortening) must be considered.

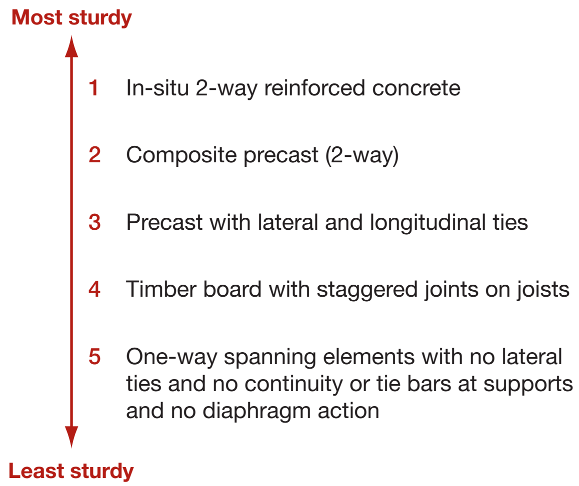


Fig 2.8 Sturdiness of floors

The great majority of infill masonry panels are designed to resist only laterally imposed loads. These panels should be adequately restrained. As a minimum this may be on two opposite sides to avoid an unrestrained corner. The methods of restraint must make due allowance for any relative movement between the masonry infill and the structural frame.

Unless the walls are designed to provide primary overall stability, it is rarely necessary to consider the influence of accidental damage to masonry infilling since its removal should not precipitate disproportionate collapse.

2.11 Openings

The size and location of openings should be such that the stability of not only the panel under consideration, but also of the adjacent walls (above and below) is not impaired.

Openings can have a major influence on the load paths in masonry walls, affecting the stress distribution. Special attention should be given to consideration of the effects of openings in the storey immediately above foundation level with regard to possible variations in bearing pressure.

The overall effect of an opening is to reduce the cross-section and, if asymmetrically disposed, to shift the centroid of the section so as to cause an overall eccentricity of the resultant of the loads (see Figure 2.9).

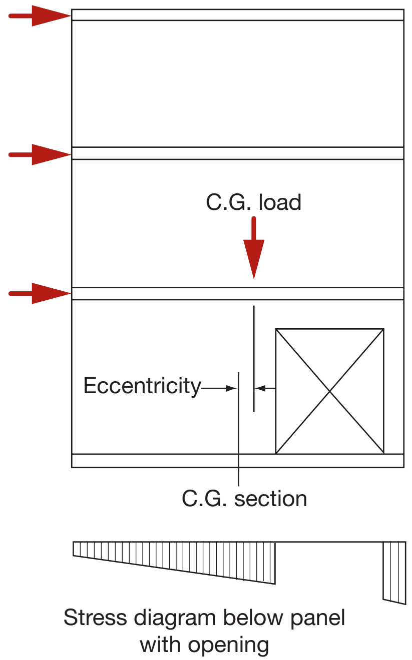
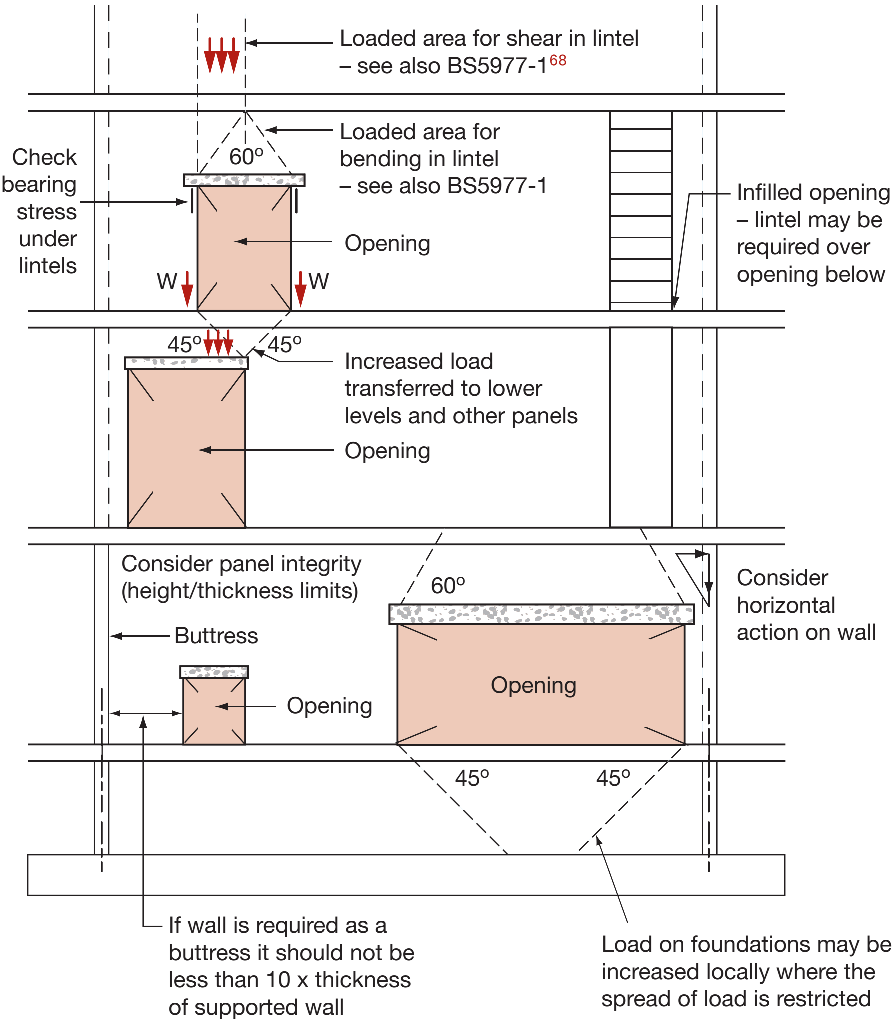


Fig 2.9 Stress arising from eccentric loading caused by the position of the opening

The local effect of an opening is to cause increased stresses under lintel bearings etc. (see Figure 2.10). Care should be exercised where openings are formed in walls that provide support or a restraining effect to an adjacent wall. (See clause 5.5.1.2 and Figure 5.1 of BS EN 1996-1-1[1]). Attention should be paid to providing suitable tying or bonding of wall junctions to achieve the desired degree of structural continuity.

New openings in existing walls require a similar evaluation in circumstances where load intensities and patterns of load in adjacent walls are subject to change. Particular consideration should be given to the effects of infilling of existing openings on the surrounding structure.



Note
The position of movement joints may influence the load spreads indicated.

Fig 2.10 Load paths arising from lintels, openings, etc.

3.1 Design co-ordination

The Engineer should be satisfied that the materials used by, and the requirements of, other members of the design team are compatible with those governing the masonry design, particularly with regard to tolerances, provisions for movement, lateral restraint, stability tying and water absorption of clay masonry units.

EC6[1] does not concern itself with resistance to impact and abrasion, or the provision of secure fixings for the attachment of other components. For example, hollow or lightweight blockwork can be less resistant to impact or abrasion and may require the use of special fixings, which could be a consideration in certain situations such as factories and warehouses.

3.2 Classification of environmental conditions

The choice of masonry materials is also governed by the exposure conditions to which they will be subjected. EC6 Part 2[3] gives a classification system for exposure conditions.

In order to give adequate consideration to the exposure of completed masonry, it is necessary to look at the climatic (macro) and local (micro) conditions to which the masonry is subject. The macro conditions which must be considered are:

- rainwater and snow
- the combination of wind and rain
- temperature variation
- relative humidity variation.

The effect of these factors on the micro conditions of exposure of the masonry must be taken into account, together with the effect of any applied finishes or protective cladding. EC6 Part 2[3] categorises the micro conditions of exposure as follows:

- MX1 - In a dry environment
- MX2 - Exposed to moisture or wetting
- MX3 - Exposed to moisture or wetting plus freeze/thaw cycling
- MX4 - Exposed to saturated salt air or seawater
- MX5 - In an aggressive chemical environment, (particularly where sulfates occur).

Where necessary, each class is sub-divided, e.g. MX2.1 and MX2.2 (see Table 3.1).

3.3 Masonry units

Masonry units should comply with the relevant BS EN Standards:

- Clay masonry units BS EN 771-1[18]
- Calcium silicate masonry units BS EN 771-2[19]
- Aggregate concrete masonry units (Dense and lightweight aggregates) BS EN 771-3[20]
- Autoclaved aerated concrete masonry units BS EN 771-4[21]
- Manufactured stone masonry units BS EN 771-5[22]
- Natural stone masonry units BS EN 771-6[23]

Where masonry units do not conform to any of the above BS ENs (e.g. reclaimed bricks), tests must be carried out to demonstrate that the units satisfy the engineering requirements.

Structural masonry units will usually be specified to achieve the compressive and/or flexural strengths required by the designer, and by reference to the exposure conditions to which they will be subject, i.e. their durability. Both strength and durability are functions of the properties of the masonry units and the mortar used. Masonry mortars are discussed in Section 3.4 and durability in Section 3.6.

Usually the designer will know the type of unit intended to be used, e.g. clay, aggregate concrete etc., but they will also need to know other details in order to complete the design to EC6. Masonry units are manufactured in one of two categories (I and II) depending on the level of quality control exercised during their manufacture. The category of the unit should be declared by the manufacturer and is needed by the designer in order to select the appropriate partial factor to be used in design (see Section 4). The designer will also need to know which group (1 to 4) the unit falls into (see Table 3.2). The grouping depends upon the percentage of voids in the unit and is required in order to calculate the characteristic compressive strength of the masonry (see Section 4). A further sub-group (1S) is used in the fire design of clay, calcium silicate and autoclaved aerated concrete structural masonry, see Section 7.5. Again, the group into which the masonry unit falls should be declared by the manufacturer. The normalised mean compressive strength of the unit is also required to calculate the characteristic compressive strength of the masonry. This may also be declared by the manufacturer, at least for the usual direction of loading (normal to the bed face).

In the case of clay units, there are two density classes, namely HD (high density) and LD (low density). The majority of clay units manufactured in the UK are of HD classification and will be the type normally used for structural masonry. In order to calculate the characteristic flexural strength of masonry made with clay units, the water absorption of the units will be required. This is already typically declared by UK producers but may not be by other producers. Clay units are also classified according to their freeze/thaw resistance and their soluble salt content, see Sections 3.6.2 and 3.6.3. Other types of units are usually declared as frost resistant, based on their performance in service. The further classification of some clay units as engineering or dpc units is related to their water absorption. The requirements for Class A and Class B clay engineering units and DPC1 and DPC2 clay damp proof course units are set out in the National Annex to BS EN 771-1: 2003[18].

Table 3.1 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 sulfates or aggressive materials.	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 sulfates 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.

Table 3.1 (Continued)

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 sulfates 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 sulfates 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 sulfates are present. Masonry in contact with highly acidic soils, contaminated ground or groundwater. Masonry near industrial areas where aggressive chemicals are airborne.

Notes

a In deciding the exposure of masonry the effect of applied finishes and protective claddings should be taken into account.

b This table is based on data from EC6 Part 2[3].

Table 3.2 Geometrical requirements for grouping of masonry units

	Materials and limits for masonry units	
	Group 1 (all materials)	Units
Volume of all holes (% of the gross volume)	⩽ 25	clay
		calcium silicate
		concrete[b]
Volume of any hole (% of the gross volume)	⩽ 12.5	clay
		calcium silicate
		concrete[b]
Declared values of thickness of webs and shells (mm)	No requirement	
		clay
		calcium silicate
		concrete[b]
Declared value of combined thickness[a] of webs and shells (% of the overall width)	No requirement	clay
		calcium silicate
		concrete[b]

Notes

a The combined thickness is the thickness of the webs and shells, measured horizontally across the unit at right angles to the face of the wall. The check is to be seen as a qualification test and need only be repeated in the case of principal changes to the design dimensions of units. The web is defined as the solid material between the holes in a masonry unit and the shell as the peripheral material between a hole and the face of a masonry unit.

Materials and limits for masonry units					
Group 2		**Group 3**		**Group 4**	
Vertical holes				**Horizontal holes**	
> 25; ⩽ 55		⩾ 25; ⩽ 70		> 25; ⩽ 70	
> 25; ⩽ 55		not used		not used	
> 25; ⩽ 60		> 25; ⩽ 70		> 25; ⩽ 50	
each of multiple holes ⩽ 2 gripholes up to a total of 12.5		each of multiple holes ⩽ 2 gripholes up to a total of 12.5		each of multiple holes ⩽ 30	
each of multiple holes ⩽15 gripholes up to a total of 30		not used		not used	
each of multiple holes ⩽ 30 gripholes up to a total of 30		each of multiple holes ⩽ 30 gripholes up to a total of 30		each of multiple holes ⩽ 25	
web	**shell**	**web**	**shell**	**web**	**shell**
⩾ 5	⩾ 8	⩾ 3	⩾ 6	⩾ 5	⩾ 6
⩾ 5	⩾ 10	not used		not used	
⩾ 15	⩾ 18	⩾ 15	⩾ 15	⩾ 20	⩾ 20
⩾ 16		⩾ 12		⩾ 12	
⩾ 20		not used		not used	
⩾ 18		⩾ 15		⩾ 45	

b In the case of conical holes, or cellular holes, use the mean value of the thickness of the webs and the shells.

c Grip holes are defined as formed voids in a masonry unit to assist handling of the unit.

d This table is based on data from EC6 Part 1-1[1].

3.4 Mortars and mortar joints

3.4.1 Masonry mortars

Masonry mortars should be selected on the grounds of strength, durability, compatibility and economy taking into account the mode of manufacture. In accordance with the National Annex[4], they may be specified as designed masonry mortars (strength performance concept) or prescribed masonry mortars (recipe concept), see Table 3.3. Both types of mortar can be either factory made masonry mortars or site-made masonry mortars. PD 6678: 2005[30] gives guidance on the specification of masonry mortars. Most mortar in the UK has previously been specified as prescribed mixes (recipe) because this is the simplest approach and produces mortars of known durability.

Designers should be aware of the possible need to test the strength and variability of masonry mortars. Although BS EN 998-2[31] in its entirety applies only to factory made mortar, it can also be referred to for site made mortars in relation to these characteristics.

The use of strong mortars with lower strength units can be associated with cracking of the units and should generally be avoided.

There is no appropriate test at European level for durability of masonry mortars. As an interim measure, when using mortars other than those listed in Table 3.3, the suitability of masonry mortar should be based on the manufacturer's experience of laboratory tests and/or actual service in masonry. Mortar may be specified for durability using the application classes defined in BS EN 998-2[31]:

- masonry subjected to passive exposure – P
- masonry subjected to moderate exposure – M
- masonry subjected to severe exposure – S.

However, there is insufficient evidence available as yet to show that this system of classification, where used by European manufacturers, conforms to UK conditions, see also Section 3.5.

Section 3.6 gives guidance on the minimum qualities of mortar required to provide adequate durability in various situations. The relationship between strength class, designation and durability is well established for the prescribed mixes given in Table 3.3 especially where the cement type comprises a high proportion of Portland cement. For other mortar prescriptions and for designed mortars the designer should be aware of the need to see that the mortar will be durable in the proposed exposure conditions.

Choice and grading of the sand has a significant effect on the consistence (workability) of the fresh mortar and on the final properties of the hardened mortar. Certain fine sands, while conforming to BS EN 13139[32], may require further adjustment of mix proportions.

Masonry cement, a mixture of Portland cement and either inert fillers or lime, sometimes incorporating admixtures, may be used in unreinforced structural masonry but see Notes **d** and **e** to Table 3.3 for limitations.

No mention of hydraulic lime mortars is made in EC6[1]. However, their use is becoming more common, particularly in repair and renovation work. Advice on their use should be sought from the manufacturer/supplier.

Plasticisers are often used to improve the consistence and durability of mortars. They do not, however, provide the extra gain of strength with time that is possible with some limes. Plasticisers should be used only in accordance with the manufacturer's instructions.

Table 3.3 Masonry mortars

	Mortar designation	Compressive strength class	Prescribed mortars (proportion of materials by volume) (see notes a and b)				Compressive strength at 28 days (N/mm^2)
			cement[c]: lime: sand with or without air entrainment	cement[c]: sand with or without air entrainment	masonry cement[d]: sand	masonry cement[e]: sand	
Increasing ability to accommodate movement, e.g. due to settlement, temperature and moisture changes. ↓	(i)	M12	1:0 to ¼:3	1:3	Not suitable	Not suitable	12
	(ii)	M6	1:½:4 to 4½	1:3 to 4	1:2½ to 3½	1:3	6
	(iii)	M4	1:1:5 to 6	1:5 to 6	1:4 to 5	1:3½ to 4	4
	(iv)	M2	1:2:8 to 9	1:7 to 8	1:5½ to 6½	1:4½	2

Notes

a Proportioning by mass will give more accurate batching than proportioning by volume, provided that the bulk densities of the materials are checked on site.

b When the sand portion is given as, for example, 5 to 6, the lower figure should be used with sands containing a higher proportion of fines whilst the higher figure should be used with sands containing a lower proportion of fines.

c Cement conforming to BS EN 197-1[24] Notation CEM I (Portland cement). Cement conforming to BS EN 197-1[24] Notation CEM II/A-S or CEM II/B-S (Portland slag cement); or CEM II/A-L or CEM II/A-LL (Portland Limestone cement); or CEM II/A-V or CEM II/B-V (Portland fly ash cement); or a combination, with equivalent proportions and properties to one of the these cements:

- Combinations produced in the mortar mixer from Portland cement CEM I conforming to BS EN 197-1[24] and ground granulated blast furnace slag conforming to BS 6699[25] where the proportions and properties conform to CEM II/A-S or CEM II/B-S of BS EN 197-1: 2000[24], except Clause 9 of that standard.
- Combinations produced in the mortar mixer from Portland cement CEM I conforming to BS EN 197-1 and limestone fines conforming to BS 7979[26] where the proportions and properties conform to CEM II/A-L or CEM II/A-LL of BS EN 197-1:2000[24], except Clause 9 of that standard.
- Combinations produced in the mortar mixer from Portland cement CEM I conforming to BS EN 197-1[24] and pulverized fuel ash conforming to BS 3892-1[27], or to BS EN 450-1[28], where the proportions and properties confirm to CEM II/A-V or CEM II/B-V of BS EN 197-1:2000[24], except Clause 9 of that standard.

d Masonry cement conforming to BS EN 413-1[29], Class MC 12.5 (inorganic filler other than lime), not less than 65% by mass of Portland cement clinker as defined in BS EN 197-1[24].

e Masonry cement conforming to BS EN 413-1[29], Class MC 12.5 (lime), not less than 65% by mass of Portland cement clinker as defined in BS EN 197-1[24].

f Table 3.3 is based on data from EC6[1] and the National Annex[4].

Care should also be taken in the use of colouring agents (pigments), to maintain compressive and bond strengths of mortars.

To avoid inconsistencies associated with site mixing it is now common practice to use dry silo mortar. The dry constituents are weight-batched at the factory and delivered to site where they are stored in a silo from which the required amounts of mortar are mixed with water and drawn off. Where silos are not practicable another option is to use retarded ready to use mortar, containing a set retarder and usually air entrainer to enable the mortar to be used over a working period of as long as 72 hours. It is also possible to obtain ready-mixed lime and sand (coarse stuff) with or without plasticiser and/or colour pigment, as may be required, although plasticiser will almost certainly be present unless otherwise specified.

Consideration should be given to minimising the number of different mortar mixes specified on a single project to reduce the risk of confusion arising on site.

Thin layer mortar is defined in BS EN 998 Part 2[31] as a designed masonry mortar with a maximum aggregate size not greater than 2mm. It is usually factory made and supplied as a dry premixed and bagged product. Typically, it has a 1:2 cement:sand (by weight) composition with admixtures. The mortar is applied to the masonry units with a serrated scoop designed to give a consistent depth of bed joint (1 to 3mm). Resin based thin layer mortars are also available which are applied by extrusion from a tube. Thin layer mortars are designed to set quickly upon application to masonry. The manufacturer must declare the minimum correction time for the mortar, i.e. the minimum period (usually 5 to 7 minutes) during which the masonry may be corrected after application of the mortar. It is very important with thin layer mortars that inappropriate additions are not introduced on site.

3.4.2 Mortar joints

Mortar joints may be finished in a number of ways. When finishing is carried out while the mortar is still fresh it is termed ‘jointing’. When the mortar is allowed to stiffen and some is then removed and replaced with fresh mortar (sometimes coloured) before finishing, or when the joint is incompletely filled initially, the process is referred to as ‘pointing’. Structurally, jointing is preferable to pointing because it leaves the bedding mortar undisturbed.

Mortar used for pointing should have mix proportions and hardened mechanical properties similar to those used in the original bedding mortar and not of a stronger designation or mix.

For all types of exposed masonry, the aim should be to fill all the joints to minimise the risk of rain penetration. Tooled and non-recessed mortar joints are more resistant to rain penetration than joints that have not been tooled, and are therefore more durable. Recessed joints increase the risk of water penetration and should be used externally only with frost-resistant units and mortars. Where used, the depth of recess should be related to the distance of any perforation or cavity in the unit from the exposed face of the unit so as to reduce the risk of water penetration.

Where the sound insulation properties of masonry are to be relied upon, it is again important that all joints are as well filled as is practicable. EC6 Part 1-1[1] recognises that it is not always necessary or possible to completely fill perpend joints, and states in Clause 8.1.5(3) ‘When units that rely on mortar pockets are used, perpend joints can be considered to be filled if mortar is provided to the full height of the joint over a minimum of 40% of the width of the unit’. For structural masonry in the UK, however, the aim should be to fill all perpend joints completely.

It is also important to avoid pointing over damp proof courses (dpcs). This provides a passage for water to bridge the dpc and may cause mortar to crumble if the dpc settles.

The principal types of joint profile used for masonry walls are shown in Figure 3.1.

Types of finish for jointing and pointing of work should be carefully chosen in relation to the durability of the units and the conditions of exposure.

Flush or bag rubbed joint
This finish gives maximum bearing area and is often favoured when coarse textured units are used. With some masonry unit types the finish may appear a little irregular.

Curved recessed (bucket handle)
This joint can give an improved appearance over a flush joint with negligible reduction in strength. It is generally considered that this joint gives the best weather resistance due to the smoothing of the joint and the superior bond this achieves. It is perhaps the most commonly used joint.

Struck or weathered
Weathered bed joints produce an interplay of light and shadow on the masonry. Such joints when correctly made have excellent strength and weather resistance.

Overhung struck
This finish gives a slightly different appearance of light and shade to struck weathered jointing. Unfortunately it allows rain to lodge on the horizontal faces of the masonry units and thus to penetrate the units and joints causing discolouration and possible front damage. For these reasons it should be confined to lightly stressed interior walls and external walls using appropriate quality units.

Square recessed
This joint, when used with durable masonry units, can produce a very pleasing effect but its weather resistance and strength will be considerably less than struck, flush or curved recessed joints. With heavily perforated units where the perforations occur near to the face, a recessed joint may be inadvisable because resistance to water penetration may be impaired.

Note
Because of their texture and frost resistance, some masonry units are unsuitable for overhung struck and square recessed jointing.

Fig 3.1 Principal types of joint profile for masonry walls

3.5 Selection of materials

Choice of masonry units should be made on the basis of information declared by the manufacturer and established suitability, demonstrated by conformity with a Technical Approval or with UK and European Standards. Where evidence of such conformity is not available, appropriate samples and testing will be required.

Table 3.4 gives details of acceptable masonry units for durability exposure classes.

Use of mortar is related in EC6 Part 2[3] to its application; the application classes (P, M or S) are related generally to the exposure of the masonry as shown in Table 3.5. Where factory made mortar is to be used in exposure classes MX4 or MX5, the manufacturer's advice should be sought regarding suitability. Selection based on existing UK practice (BS 5628 Part 3[8]) is discussed in Section 3.6.1.

Table 3.4 Acceptable specifications of masonry units for durability

Exposure class	Clay masonry units[a] conforming to BS EN 771-1[18]	Calcium silicate masonry units conforming to BS EN 771-2[19]	Aggregate concrete masonry units conforming to BS EN 771-3[20]		Autoclaved aerated concrete masonry units conforming to BS EN 771-4[21]	Manufactured stone masonry units conforming to BS EN 771-5[22]	Natural stone masonry units conforming to BS EN 771-6[23]
			Dense aggregate	Lightweight aggregate			
MX1[b]	Any	Any	Any	Any	Any	Any	Any
MX2.1	F0, F1 or F2 / S1 or S2	Any	Any	Any	Any	Any	Any
MX2.2	F0, F1 or F2 / S1 or S2	Any	Any	Any	$\geqslant$ 400 kg/m^3	Any	Any
MX3.1	F1 or F2 / S1 or S2	Freeze/thaw[c] resistant	Freeze/thaw[c] resistant	Freeze/thaw[c] resistant	$\geqslant$ 400 kg/m^3	Any	Consult manufacturer
MX3.2	F2, S1 or S2	Freeze/thaw[c] resistant	Freeze/thaw[c] resistant	Freeze/thaw[c] resistant	$\geqslant$ 400 kg/m^3	Any	Consult manufacturer
MX4	In each case assess the degree of exposure to salts, wetting and freeze/thaw cycling and consult the manufacturer.						
MX5	In each case a specific assessment should be made of the environment and the effect of the chemicals involved taking into account concentrations, quantities available and rates of reaction and consult the manufacturer.						

Notes

a For explanation of F0, F1, F2, S1 and S2 see Sections 3.6.2 and 3.6.3.

b Class MX1 is valid only as long as the masonry, or any of its components, is not exposed during execution to more severe conditions over a prolonged period of time.

c See Table 3.7 for guidance on durability.

d This table is based on data from EC6 Part 2[3].

Table 3.5 Acceptable specifications of mortars for durability

Exposure class	Mortar in combination with any type of unit, classified according to Section 3.4.1
MX1[a,b]	P, M or S
MX2.1	M or S
MX2.2	M or S[c]
MX3.1	M or S
MX3.2	S[c]
MX4	In each case assess the degree of exposure to salts, wetting and freeze/thaw cycling and consult the manufacturers of the constituent materials.
MX5	In each case a specific assessment should be made of the environment and the effect of the chemicals involved taking into account concentrations, quantities available and rates of reaction and consult the manufacturers of the constituent materials.

Notes

a Class MX1 is valid only as long as the masonry, or any of its components, is not exposed during execution to more severe conditions over a prolonged period of time.

b When designation P mortars are specified it is essential to ensure that masonry units, mortar and masonry under construction are fully protected from saturation and freezing.

c When clay masonry units of Soluble Salts Contents Category S1 are to be used in masonry where the Exposure Class is MX2.2, MX3.2, MX4 or MX5 the mortars should in addition be sulfate resisting.

d This table is based on data from EC6 Part 2[3].

Table 3.6 Selection of ancillary components in relation to material/coating specification and situation

Material/ coating reference in accordance with BS EN 845-1[33]	Material/ coating reference in accordance with BS EN 845-2[34]	Material/ coating reference in accordance with BS EN 845-3[35]	Materials/coating specification[a]
1		R1	Austenitic stainless steel (molybdenum chrome nickel alloys)
2			Plastic used for the body of ties
3	L3	R3	Austenitic stainless steel (chrome nickel alloys)
4			Ferritic stainless steel
5, 6, 7			Aluminium bronze, phosphor bronze, copper
8, 9			Zinc coated (940g/m²) steel wire or component
10	L10		Zinc coated (710g/m²) steel component
11	L11		Zinc coated (460g/m²) steel component
	L11.1		Zinc coated (460g/m²) steel component with organic coating over all outer surfaces of finished component
	L11.2		
12.1	L12.1		Zinc coated (300g/m²) steel strip or sheet with organic coating over all outer surfaces of finished component
12.2	L12.2		
13		R13	Zinc coated (265g/m²) steel wire
14	L14		Zinc coated (300g/m²) steel strip or sheet with all cut edges organic coated
15			Zinc pre-coated (300g/m²) steel strip or sheet
16.1	L16.1		Zinc coated (137g/m²) steel strip or sheet with organic coating over all outer surfaces of finished component
16.2	L16.2		
17			Zinc pre-coated (137g/m²) steel strip with zinc coated edges
18		R18	Zinc coated (60g/m²) steel wire with organic coating over all surfaces of finished component
19		R19	Zinc coated (105g/m²) steel wire
20		R20	Zinc coated (60g/m²) steel wire
21		R21	Zinc pre-coated (137g/m²) steel sheet

	Situation in buildings up to three storeys in a non-aggressive environment (For other buildings see Notes b and c)											
	In contact with or embedded in an internal wall or an inner leaf of an external cavity wall Exposure class MX1						In contact with or embedded in an outer leaf of an external cavity wall or a single leaf external wall Exposure classes MX2/3					
	T	S	L	L+	R	C	T	S	L	L+	R	C
	Y	Y			Y	Y	Y	Y			Y	Y
	Y						Y					
	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
	X	X				X	X	X				X
	Y	Y				Y	Y	Y				Y
	Y	Y				Y	Y	Y				Y
	X	Y	Y	Y		Y	X	X	Y	Y		X
	X	Y	X	Y		Y	X	X	X	Y		X
			Y	Y					Y	Y		
			Y	Y					Y	Y		
	X	Y	Y	Y		Y	X	X	Y	Y		X
	X	Y	Y	Y		Y	X	X	Y	Y		X
	X	Y			Y	Y	X	X			X	X
	X	Y	X	Y		Y	X	X	X	Y		X
	X	Y				Y	X	X				X
	X	Y	X	Y		Y	X	X	X	Y		X
	X	Y	Y	Y		Y	X	X	Y	Y		X
	X	Y				Y	X	X				X
	X	X			X	X	X	X			X	X
	X	X			X	X	X	X			X	X
	X	X			X	X	X	X			X	X
	X	X			Y	X	X	X			X	X

Table 3.6 (Concluded)

Key to product types

- T Ties and brackets conforming to BS EN 845-1[33] with this material/coating reference.
- S Tension straps and hangers conforming to BS EN 845-1[33] with this material/coating reference.
- L Lintels conforming to BS EN 845-2[34] with this material/coating reference have integral damp proof systems and may be used without the need to provide a separate damp proof course.
- L+ Lintels conforming to BS EN 845-2[34] with this material/coating reference may be used in these locations with a separate damp proof course.
- R Bed joint reinforcement conforming to BS EN 845-3[35] with this material/coating reference.
- C Components outside the scope of BS EN 845-1 to 3[33-35] but meeting the specification requirements of BS EN 845-1[33] for the material/coating reference.

Key to table

- Y The use of components with this material/coating reference may be used in these locations.
- (Blank) Components with this material/coating reference are not specified.
- X The use of components with this material/coating reference is not recommended in these locations.

Notes

a See BS EN 845-1 to 3[33-35] for the full details of the specifications. The zinc coating weights are approximate values for one surface.

b In contact with or embedded in outer leaf of an external cavity wall or a single leaf external wall, in buildings exceeding three storeys in a non aggressive environment, the material/coating specifications should be limited to austenitic stainless steels (references 1, R1, 3, L3 and R3) or aluminium bronze, phosphor bronze or copper (references 5, 6 or 7).

c For buildings located in an aggressive environment (e.g. exposure classes MX4 and 5) specialist advice should be sought.

d This table is based on data from BS 5628 Part 3[8].

3.6 Durability

3.6.1 General

For masonry construction the requirements for durability are usually satisfied by the appropriate choice of material qualities and mortar-joint profile, together with an adequate standard of workmanship. The minimum qualities of units and mortar that are considered to provide adequate durability are given in Tables 3.4 and 3.5. Guidance on the selection of ancillary components, wall ties etc. in relation to durability is given in Table 3.6, see also Section 3.7.

Existing UK practice (BS 5628 Part 3[8]) relates durability of masonry to unit type and mortar designation, see Table 3.7. Whilst Table 3.3 implies an equivalence between designation and strength class, in terms of durability it should not be assumed that it is sufficient to specify by strength class alone. If a designation is not specified, then the exposure class of the mortar should be specified, see Table 3.5.

3.6.2 Frost resistance

As well as temperature, water saturation is a major factor adversely affecting frost resistance. If freeze/thaw cycles occur, saturated masonry will be liable to frost failure unless appropriate units and mortar are specified.

When appropriate, manufacturers of masonry units are required to declare the freeze/thaw resistance of their units.

Until a European method of test is available, the freeze-thaw resistance of clay units may be evaluated on provisions valid in the place of use and classified as:

- F0 - Passive exposure
- F1 - Moderate exposure, and
- F2 - Severe exposure.

Masonry should preferably be detailed so that the risk of saturation, particularly for exposed locations, is reduced by adopting details that throw water clear of the walls, e.g. by the use of copings, sills, and roofs with adequate overhangs and drips. The provision of damp proof courses is also important.

Some architectural features, such as flush copings and sills, can result in masonry being exposed and saturated locally. For these features it is essential to select masonry units and mortar of appropriate durability.

Table 3.7 Durability of masonry in finished construction

Masonry condition or situation		Quality of masonry units and appropriate mortar designations		
		Clay units	**Calcium silicate unit**	**Concrete bricks**
(A) Work below or near external ground level				
A1	Low risk of saturation	Without freezing LD - F0 and S0 or HD - F0, F1 or F2 and S0, S1 or S2 in (i), (ii) or (iii) With freezing HD - F1 or F2 and S0, S1 or S2 in (i), (ii) or (iii)	Without or with freezing, compressive strength class 20 or above in (iii) or (iv) (see remarks)	Without or with freezing $\geq$ 16.5N/mm^2 in (iii)
A2	High risk of saturation without freezing	HD - F1 or F2 and S1 and S2 in (i) or (ii) (see remarks)	Compressive strength class 20 or above in (ii) or (iii)	$\geq$ 16.5N/mm^2 in (ii) or (iii)
A3	High risk of saturation with freezing	HD - F2 and S1 or S2 in (i) or (ii) (see remarks)	Compressive strength class 20 or above in (ii)or (iii)	$\geq$ 22N/mm^2 in (ii) or (iii)
(B) Masonry DPCs				
B1	In buildings	DPC 1 units to BS EN 771-1[18] in (i)	Not suitable	Not suitable
B2	In external works	DPC 1 or 2 to BS EN 771-1[18] in (i)	Not suitable	Not suitable
(C) Unrendered external walls (other than chimneys, cappings, copings, parapets, sills)				
C1	Low risk of saturation	HD - F1 or F2 and S1 or S2 in (i), (ii) or (iii)	Compressive strength class 20 or above in (iii) or (iv) (see remarks)	$\geq$ 7.3N/mm^2 in (iii)
C2	High risk of saturation	HD - F2 and S1 or S2 in (i) or (ii) (see remarks)	Compressive strength class 20 or above in (iii)	$\geq$ 18N/mm^2 in (iii)

Note This table is based on data from BS 5628 Part 3[8]

	Concrete blocks	Remarks
	Without or with freezing a) of net density ⩾ 1500kg/m^3; or b) made with dense aggregate; or c) having a compressive strength ⩾ 7.3N/mm^2; or d) most types of autoclaved aerated block (see remarks) All in (iii) or (iv) (see remarks)	Some types of autoclaved aerated concrete block may not be suitable. The manufacturer should be consulted. If sulfate ground conditions exist, consideration should be given to the use of strong mortars using Portland cement or using sulfate-resisting Portland cement. Where designation (iv) mortar is used it is essential to ensure that all masonry units, mortar and masonry under construction are protected fully from saturation and freezing.
	As for A1 in (ii) or (iii)	The masonry most vulnerable in A2 and A3 is located between 150mm above, and 150mm below finished ground level. In this area masonry will become wet and can remain wet for long periods of time, particularly in winter. Where S1 clay units in designation (ii) mortar are used in A2 or A3, sulfate-resisting Portland cement should be used in the mortar. In conditions of highly mobile groundwater, consult the manufacturer on the selection of materials.
	As for A1 in (ii)	
	Not suitable	Masonry DPCs can resist rising damp but will not resist water percolating downwards. If sulfate ground conditions exist, consideration should be given to the use of strong mortars using Portland cement or using sulfate-resisting Portland cement. DPCs of clay units are unlikely to be suitable for walls of other masonry units, as differential movement can occur.
	Not suitable	
	Any in (iii) or (iv) (see remarks)	Walls should be protected by roof overhang and other projecting features to minimise the risk of saturation. However, weathering details may not protect walls in conditions of very severe driving rain. Certain architectural features, e.g. masonry below large glazed areas with flush sills, increase the risk of saturation. Where designation (iv) mortar is used it is essential to ensure that all masonry units, mortar and masonry under construction are protected fully from saturation and freezing. Where S1 clay units are used in designation (ii) mortar for C2, sulfate–resisting Portland cement should be used in the mortar.
	Any in (iii)	

Table 3.7 (Continued)

Masonry condition or situation		Quality of masonry units and appropriate mortar designations		
		Clay units	**Calcium silicate unit**	**Concrete bricks**
(D) Rendered external walls (other than chimneys, cappings, copings, parapets, sills)				
D1	Rendered external walls	HD - F1 or F2 and S1 or S2 in (i), (ii) or (iii) (see remarks)	Compressive strength class 20 or above in (iii) or (iv) (see remarks)	⩾ 7.3N/mm² in (iii)
(E) Internal walls and inner leaves of cavity walls above dpc level				
E1	Internal walls and inner leaves of cavity walls	LD - F0 and S0 or HD - F0, F1 or F2 and S0, S1 or S2 in (i), (ii), (iii) or (iv) (see remarks)	Compressive strength class 20 or above in (iii) or (iv) (see remarks)	> 7.3N/mm² in (iii) or (iv) (see remarks)
(F) Unrendered parapets (other than cappings and copings)				
F1	Low risk of saturation e.g. low parapets on some single–storey buildings	HD - F1 or F2 and S1 or S2 in (i), (ii) or (iii)	Compressive strength class 20 or above in (iii)	⩾ 24N/mm² in (iii)
F2	High risk of saturation e.g. where a capping only is provided for the masonry	HD - F2 and S1 or S2 in (i) or (ii) (see remarks)	Compressive strength class 20 or above in (iii)	⩾ 24N/mm² in (iii)
(G) Rendered parapets (other than cappings and copings)				
	Rendered parapets	HD - F1 or F2 and S2 in (i), (ii) or (iii) HD - F1 or F2 and S1 in (i) or (ii) (see remarks)	Compressive strength class 20 or above in (iii)	⩾9N/mm² in (iii)

Note This table is based on data from BS 5628 Part 3[8]

	Concrete blocks	Remarks
	Any in (iii) or (iv) (see remarks)	Rendered walls are usually suitable for most wind-driven rain conditions. Where S1 clay units are used, sulfate–resisting Portland cement should be used in the jointing mortar and in the base coat of the render. Clay units of F1/S1 designation are not recommended for the rendered outer leaf of a cavity wall with full-fill insulation. Where designation (iv) mortar is used it is essential to ensure that all masonry units, mortar and masonry under construction are protected fully from saturation and freezing.
	Any in (iii) or (iv) (see remarks)	Where designation (iv) mortar is used it is essential to ensure that all masonry units, mortar and masonry under construction are protected fully from saturation and freezing.
	a) of net density $\geqslant$ 1500kg/m^3; or b) made with dense aggregate; or c) having a compressive strength $\geqslant$ 7.3N/mm^2; or d) most types of autoclaved aerated block (see remarks) in (iii)	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. Where S1 clay units are used in F2, sulfate-resisting Portland cement should be used in the mortar.
	As for F1 in (ii)	
	Any in (iii)	Single-leaf walls should be rendered only on one face. All parapets should be provided with a coping. Where S1 clay units are used, sulfate-resisting Portland cement should be used in the jointing mortar and in the base coat of the render.

Table 3.7 (Continued)

Masonry condition or situation		**Quality of masonry units and appropriate mortar designations**			
		Clay units	**Calcium silicate unit**	**Concrete bricks**	
(H) Chimneys					
H1	Unrendered with low risk of saturation	HD - F1 or F2 and S1 and S2 in (i), (ii) or (iii) (see remarks)	Compressive strength class 20 or above in (iii) (see remarks)	⩾12N/mm² in (iii) (see remarks)	
H2	Unrendered with high risk of saturation	HD - F2 and S1 or S2 in (i) or (ii) (see remarks)	Compressive strength class 20 or above in (iii) (see remarks)	⩾ 18N/mm² in (iii) (see remarks)	
H3	Rendered	HD - F1 or F2 and S2 in (i), (ii) or (iii) or HD - F1 or F2 and S1 in M12, or M6 (see remarks)	Compressive strength class 20 or above in (iii) (see remarks)	⩾ 9N/mm² in (iii) (see remarks)	
(I) Cappings, copings and sills					
	Cappings, copings and sills	HD - F2 and S1 or S2 in (i)	Compressive strength class 30 or above in (ii)	⩾ 36N/mm² in (ii)	

Note This table is based on data from BS 5628 Part 3[8]

	Concrete blocks	**Remarks**
	Any in (iii)	Chimney stacks are normally the most exposed masonry on any building. Because of the possibility of sulfate attack from flue gases the use of sulfate-resisting Portland cement in the jointing mortar and in any render is strongly recommended. Masonry with tile cappings cannot be relied upon to keep out moisture. The use of a coping is preferable. Some types of autoclaved aerated concrete block may not be suitable for use in H2. The manufacturer should be consulted.
	a) of net density ⩾ 1500kg/m³; or b) made with dense aggregate; or c) having a compressive strength ⩾ 7.3N/mm²; or d) most types of autoclaved aerated block (see remarks) in (ii)	
	Any in (iii)	
	a) of net density ⩾ 1500kg/m³; or b) made with dense aggregate; or c) having a compressive strength ⩾ 7.3N/mm² in (ii)	Autoclaved aerated concrete blocks are not suitable for use in I. Where cappings or copings are used for chimney terminals, the use of sulfate–resisting Portland cement is strongly recommended. DPCs for cappings, copings and sills should be bedded in the same mortar as the masonry units.

3.6.3 Soluble salt content

Most clays used in brickmaking contain soluble salts that have been retained in the fired bricks. If the masonry becomes saturated for long periods, these soluble salts may become mobile in solution and may contain soluble sulfates. These may cause mortars that have been incorrectly specified or batched and have a low Portland cement content to deteriorate under sulfate attack. Sulfates from the ground or other sources may be equally destructive.

Manufacturers of clay masonry units are required to declare the active soluble salts category of their bricks in relation to their maximum content of water soluble sulfates, as S0, S1 or S2. The use of category 'S2' rather than 'S1' or 'S0' units in clay masonry that may remain saturated for long periods should reduce the risk of sulfate attack in the mortar.

Sulfate attack can also occur on mortar joints and on concrete masonry units in the presence of external sulfates, e.g. those dissolved in groundwater.

3.6.4 Efflorescence

Efflorescence is a crystalline or glassy deposit left on the surface of clay and concrete masonry after the evaporation of water carrying dissolved soluble salts. It is a harmless occurrence and, whilst unsightly, usually of a temporary nature. The risk of efflorescence can best be minimised by providing protection from rain to the masonry units in stacks and to newly built masonry.

3.6.5 Lime bloom

Lime bloom is a white stain occurring on concrete or mortar surfaces or surfaces in close proximity to concrete units caused by lime being leached out of the unit. Lime bloom is not like efflorescence and is not soluble in water but can be removed by careful washing with an appropriate acid. Unprotected masonry built in wet weather is sometimes susceptible to this form of staining.

3.7 Ancillary components

Ancillary components are covered by BS EN 845[33-35]. Part 1[33] deals with ties, tension straps, hangers and brackets, Part 2[34] with lintels and Part 3[35] with bed joint reinforcement. In each Part the information to be provided by the manufacturer, e.g. material/coating specification, dimensions, declared strengths and specification for use, is listed.

Guidance on protection against corrosion for ancillary components such as ties, brackets and lintels is given in EC6 Part 2[3], but as, to some extent, this does not reflect UK practice, the guidance given in Table 3.6 should be used.

3.8 Damp proof courses

Despite the widespread use of damp proof courses (dpcs) in masonry elements, their structural properties, particularly in tension, have not been widely studied. EC6 Part 1[1] simply requires that damp proof courses shall resist the passage of water.

The principal factors to be considered are:

- resistance to squeezing out due to compressive loads
- ability to resist sliding and/or shear stresses
- adhesion to mortar so that flexural stresses may be transmitted.

In general, advice on the resistance to compression, tension, sliding and shear should be sought from the manufacturers. In particular it should be noted that the flexural strength of damp proof courses should not be relied upon unless suitable evidence based on test data is available. Commonly used damp proof course materials and their properties are listed in Table 3.8.

Where flexural or shear strength is important, consideration may be given to the use of a clay masonry unit damp proof course, see Section 3.3.

Table 3.8 Physical properties and performance of materials for dpcs

Material	Minimum mass (kg/m²)	Minimum thickness (mm)	Joint treatment to prevent water moving	
			Upward	Downward
(A) Flexible				
Lead sheet	[a]	1.8	Lapped at least 100mm	100mm passing lap and interlocking upstand
Copper conforming approximately to BS EN 1172[37], 2.28 grades C104 or C106 in the O condition	Approximately 2.28	0.25	Lapped at least 100mm	Welded or welted
Bitumen hessian base (class A of BS 6398[38])	3.8		Lapped at least 100mm	Lapped at least 100mm and sealed
Bitumen fibre base (class B of BS 6398[38])	3.3			
Bitumen hessian base and lead (class A of BS 6398[38])	4.4			
Bitumen fibre base and lead (class E of BS 6398[38])	4.4			
Polyethylene, low density ($0.915 g/cm^3$ to $0.925 g/cm^3$) complying with BS 6515[39]	Approximately 0.5	0.46	Seal with welted double-sided adhesive tape	Welted
Bitumen polymer and pitch polymer	Approximately 1.5	1.10	Lapped at least 100mm	Lapped at least 100mm and sealed

Notes

a Code number 4 in BS 743[36]

b This table is based on data from BS 5628 Part 3[8]

	Liability to extrusion	Durability	Other consideration
	Not under pressure met in normal construction	Corrodes in contact with mortars. Protect with bitumen or bitumen paint of heavy consistency applied to both surfaces of the lead.	Easily worked to required shape but this is a slow process. Limit lengths to 1.5m.
	Not under pressure met in normal construction	Highly resistant to corrosion. If soluble salts are present, protected as for lead.	Can stain masonry. Not easy to work on site, so not suitable for cavity trays. Avoid contact with aluminium.
	Likely to extrude under heat and moderate pressure but this is unlikely to affect resistance to moisture penetration	The hessian or fibre can decay but this does not affect efficiency if the bitumen remains undisturbed.	Materials should be unrolled with care. In cold weather, warm before use. When used as a cavity tray, the dpc should be fully supported. For further guidance see BS 6398, Annex B[38].
	Not under pressure met in normal construction	No evidence of deterioration in contact with other building materials.	Accommodates considerable lateral movement. When used as a cavity tray, can be difficult to hold in place and may need bedding in mastic for the full thickness of the outer leaf, to prevent rain penetration. Not suitable for use where compressive stress is low, e.g. under copings.
	Not under pressure met in normal construction	Unlikely to be impaired by any construction movements normally occurring up to the point of failure of the wall.	Accommodates considerable lateral movement. When used as a cavity tray, pre-formed cloaks should be used, e.g. at changes of level and junctions.

Table 3.8 Continued

Material	Minimum mass (kg/m²)	Minimum thickness (mm)	Joint treatment to prevent water moving		
			Upward	Downward	
(B) Semi-rigid					
Mastic asphalt conforming to BS 6925[40] of hardness appropriate to conditions		12	No joint problems	No joint problems	
(C) Rigid					
Clay DPC units of prescribed water absorption conforming to BS EN 771-1[18]		Two courses, laid to break joint, bedded in 1:3 Portland cement: sand mortar	No joint problems	Not suitable	
Slate conforming to BS 743[36] and most granites		Two courses laid to break joint, bedded in 1:3 Portland cement: sand mortar	No joint problems	Not suitable	

	Liability to extrusion	Durability	Other consideration
	Liable to extrude under pressure above 0.65N/mm^2	No deterioration	To provide a key for mortar below next course of masonry, up to 35% grit should be beaten into asphalt immediately after application and left proud of surface. Alternatively, the surface should be scored whilst still warm.
		No deterioration	Particularly appropriate where dpc is required to transmit tension, e.g. at base of a freestanding wall. DPC Designation I units are required for buildings. DPC Designation 1 or 2 may be used in external works (see BS EN 771-1 Clause NA.3.10[18]).
		No deterioration	

4.1 Loading

The loads (actions) to be used in calculations are:

- Characteristic dead load (permanent action) G_k. The weight of the structure complete with finishes, fixtures and fixed partitions.
- The characteristic live loads (variable actions) Q_k. Where only one variable action is being considered in the combination of the effects of actions, this action is chosen to be $Q_{k,1}$. When a number of variable actions act simultaneously, the leading variable action is chosen as $Q_{k,1}$ and the other actions (accompanying) are chosen as $Q_{k,i}$ and are reduced by an appropriate combination factor. Where it is not obvious which should be the leading variable action, each action should be checked in turn and the worst case taken.
- Accidental loading (see Appendix B).

For typical buildings these loads are found in the following Eurocodes and their respective National Annexes (NA):

- BS EN 1991: Eurocode 1: Actions on structures (EC1)
  - Part 1-1: General actions - Densities, self-weight and imposed loads[41,42]
  - Part 1-3: General actions - Snow loads[43,44]
  - Part 1-4: General actions - Wind loads[45,46]
  - Part 1-7: General actions - Accidental actions from impact and explosions[47,48]
- BS EN 1997: Eurocode 7: Geotechnical design (EC7)[49]

Imperfections e.g. out of plumb should be taken into account in design. Thus at the ultimate limit state the horizontal forces to be resisted at any level should be the sum of:

(i) The horizontal load due to the vertical load being applied to a structure with a notional inclination. This notional inclination is taken as

$$\text{angle } \upsilon = \frac{1}{\left(100\sqrt{h_{tot}}\right)} \text{ radians to the vertical.}$$

Where: h_{tot} is the total height of the structure in metres.

This notional inclination leads to all vertical actions having a corresponding horizontal action. This horizontal action should have the same load factor and combination factor as the vertical load it is associated with.

(ii) The wind load derived from the NA to BS EN 1991-1-4[46], multiplied by the appropriate partial factor.

The horizontal forces should be distributed between the resisting elements according to their stiffness.

Current UK practice is for the strongpoints (shear walls) providing overall stability to the structure to be designed for a lateral load being the maximum of 1.5% of the characteristic dead load or the design wind load. Thus, whilst UK practice uses a constant proportion of the dead load *or* the wind load to design for overall stability, EC6 uses a smaller proportion of the total load (the proportion reducing with building height) combined with the wind load.

4.2 Limit states

This *Manual* adopts the limit state principle and the partial factor format common to all Eurocodes and defined in BS EN 1990[51].

4.2.1 Ultimate limit state

The design loads are obtained by multiplying the characteristic loads by the appropriate partial factor γ_G, γ_Q from Table 4.1 which gives the UK National Annex to EC0[52] values.

When more than one live load (variable action) is present the secondary live load may be reduced by the application of a combination factor ψ_0 see Table 4.2.

The basic load combination for a typical building structure becomes:

$$\gamma_G\, G_k + \gamma_Q\, Q_{k,1} + \Sigma\, \gamma_Q\, \psi_0\, Q_{k,i}$$

Where $Q_{k,1}$ and $Q_{k,i}$ ($Q_{k,2}$, $Q_{k,3}$ etc.) are the actions due to vertical imposed loads, wind loads and snow etc., $Q_{k,1}$ being the leading action for the situation considered. BS EN 1990[51] allows alternative combinations which whilst more complex may allow for greater economy.

The 'unfavourable' and 'favourable' factors should be used so as to produce the most onerous condition. Generally permanent actions from a single load source may be multiplied by either the 'unfavourable' or the 'favourable' factor. For example, all actions originating from the self weight of the structure may be considered as coming from one source and there is no requirement to consider unfavourable and favourable factors on different spans. Exceptions to this are where overall equilibrium is being checked and the structure is very sensitive to variations in permanent loads.

4.2.2 Serviceability limit states

The appropriate serviceability limit state may be considered for each specific case. However, EC6 Part 1-1[1] adopts a similar approach to that of BS 5628 Part 1[7] in relation to plain masonry, in that it considers that if the ultimate limit state has been satisfied, then no specific checks for serviceability limit states are generally required. Where checks are necessary, appropriate load cases are given in Table 4.3 and are obtained by multiplying the characteristic variable actions by appropriate reduction factors (ψ_1 or ψ_2). The values of ψ_1 and ψ_2 are given in Table 4.2 which gives UK National Annex to EC0[52] values.

Table 4.1 Partial factors for actions γ at the ultimate limit state				
Permanent Action (Dead load) G_k		**Variable Actions (Imposed, wind and snow load) $Q_{k,i}$**		**Earth[b] and water[c] (these can generally be considered as permanent actions and factored accordingly)**
$\gamma_{G,sup}$	$\gamma_{G,inf}$	γ_Q (unfav)	γ_Q (fav)	
1.35	**1.00**	**1.50**	**0.00**	**1.35**

Notes

a Alternative values may be required to check overall equilibrium of structures sensitive to variation in dead weight (see BS EN 1990[51]).

b This assumes that combination 1 of Case 1 (see EC7[49]) is critical for the structural design. This is normal for typical foundations when sized to EC7.

c If the water pressure calculated is the most unfavourable value that could occur during the life of the structure a partial factor of 1.0 may be used.

d This table is based on data from the NA[52] to EC0.

Table 4.2 Factors for buildings			
Action	ψ_0	ψ_1	ψ_2
Domestic, residential area	**0.7**	**0.5**	**0.3**
Office area	**0.7**	**0.5**	**0.3**
Congregation areas	**0.7**	**0.7**	**0.6**
Shopping areas	**0.7**	**0.7**	**0.6**
Storage areas	**1.0**	**0.9**	**0.8**
Traffic area Vehicle $\leqslant$ 30kN	**0.7**	**0.7**	**0.6**
Traffic area 30kN < Vehicle $\leqslant$ 160kN	**0.7**	**0.5**	**0.3**
Roofs	**0.7**	**0.0**	**0.0**
Snow loads H > 1000m above sea level	**0.7**	**0.5**	**0.2**
Snow loads H $\leqslant$ 1000m above sea level	**0.5**	**0.2**	**0.0**
Wind loads	**0.5**	**0.2**	**0.0**
Temperature (non fire)	**0.6**	**0.5**	**0.0**

Note

This table is based on data from the NA[52] to EC0.

Table 4.3 Serviceability load cases			
Combination	**Permanent Actions**	**Variable Actions**	
	$G_{k,sup}$	**Leading** $Q_{k,1}$	**Others** $Q_{k,i}$
Characteristic	1.0	1.0	ψ_0
Frequent	1.0	ψ_1	ψ_2
Quasi-permanent	1.0	ψ_2	ψ_2

Note
This table is based on data from the NA[52] to EC0.

4.3 Characteristic strengths

4.3.1 General

These are taken as the value of the strength of masonry having a probability of 5% of not being attained in a series of tests (for practical purposes, not less than five in number). In some cases, a nominal value is used where a suitable statistical basis is unavailable.

4.3.2 Characteristic compressive strength of masonry

This is taken as the characteristic strength of masonry in compression without the effects of platen restraint, slenderness or eccentricity of loading. It is determined either from:

i) The results of tests carried out in accordance with BS EN 1052-1[53] *Methods of test for masonry - Determination of compressive strength*, which may be tabulated or expressed in terms of a generalised formula for the relationship between the characteristic compressive strength of masonry, the normalised mean compressive strength of the unit and the mortar strength. The test results may be available from a database or from tests carried out specifically for the project, although this is rarely likely to be economically viable, or

ii) Using similar formulae but with specific constants for various types of masonry and specific limitations on values of normalised compressive strength etc.

In order to be able to modify the limitations imposed by method ii) to suit UK requirements, the National Annex[4] adopts method i) using the formula:

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

Where: f_k is the characteristic compressive strength of masonry

K is a constant, the value of which is obtained from the National Annex[4] to EC6 Part 1-1, see Table 4.4

f_b is the normalised mean compressive strength of the unit as obtained by test in accordance with BS EN 772-1[54] or from the manufacturer

f_m is the compressive strength of the mortar. It may be taken as equal to the mortar strength class

α and β are powers, the values of which are obtained from the National Annex to EC6 Part 1-1[4], see Table 4.5.

Table 4.4 Values of K for use with general purpose, thin layer and lightweight mortars

Masonry unit		General purpose mortar	Thin layer mortar (bed joint $\geqslant$ 0.5mm and $\leqslant$ 3mm)	Lightweight mortar of density	
				$600 \leqslant \rho_d \leqslant 800 kg/m^3$	$800 < \rho_d \leqslant 1500 kg/m^3$
Clay	Group 1	**0.50**	**0.75**	**0.30**	**0.40**
	Group 2	**0.40**	**0.70**	**0.25**	**0.30**
	Group 3	a	a	a	a
	Group 4	a	a	a	a
Calcium silicate	Group 1	**0.50**	**0.80**	b	b
	Group 2	**0.40**	**0.70**	b	b
Aggregate concrete	Group 1	**0.55**	**0.80**	**0.45**	**0.45**
	Group 1[c] (units laid flat)	**0.50**	**0.70**	**0.40**	**0.40**
	Group 2	**0.52**	**0.76**	**0.45**	**0.45**
	Group 3	a	a	a	a
	Group 4	a	a	a	a
Autoclaved aerated concrete	Group 1	**0.55**	**0.80**	**0.45**	**0.45**
Manufactured stone	Group 1	**0.45**	**0.75**	b	b
Dimensioned natural stone	Group 1	**0.45**	b	b	b

Notes

a Group 3 and 4 units have not traditionally been used in the UK, so no values are available.

b These masonry unit and mortar combinations have not been traditionally used in the UK, so no values are available.

c If group 1 aggregate concrete units contain formed vertical voids, multiply K by **(100 - n)/100**, where n is the percentage of voids, maximum 25%.

d See Table 3.2 for definition of groups.

e This table is based on data from the NA[4] to EC6 Part 1-1.

The normalised mean compressive strength f_b of a masonry unit is derived from the unit compressive strength obtained from tests in accordance with BS EN 772-1[54]. The tests are usually performed on units placed in their normal (upright) orientation (see 'Further limitations' in Table 4.5). This *Manual* recommends that perpend joints should always be fully filled (within practical limits); however, EC6 allows that the equations in Table 4.5 may be used where perpend joints are unfilled, providing consideration is given to any horizontal loads involved. The UK National Annexes for EC6 Part 1-1[4] and Part 1-2[5] do not give any design data for Group 3 and Group 4 units because no UK data base exists at present to calibrate K adequately.

It may be noted that the limitations on joint thickness for the thin layer mortar equations in Table 4.5 make the value of f_k independent of f_m.

4.3.3 Characteristic shear strength of masonry

The characteristic shear strength of masonry f_{vk} depends upon the characteristic initial shear strength of the masonry f_{vk0} (obtained from tests, Table 3.4 of EC6 Part 1-1[1], or national database) and the design compressive stress perpendicular to the shear plane, at the level concerned, σ_d.

The characteristic shear strength is given by:

$f_{vk} = f_{vk0} + 0.4\ \sigma_d$ for fully filled joints

$f_{vk} = 0.5\ f_{vk0} + 0.4\ \sigma_d$ for unfilled perpend joints

Where: f_{vk0} is the characteristic initial shear strength, under zero compressive strength

σ_d is the design compressive stress perpendicular to the shear in the member at the level under consideration using the appropriate load combination based on the average vertical stress over the compressed part of the wall that is providing shear resistance.

The upper limit of the characteristic shear strength is $\mathbf{0.065}\boldsymbol{f_b}$ which is the value adopted in the UK National Annex[4] to EC6 Part 1-1 for masonry with fully filled joints. For masonry in which the perpend joints are unfilled, the upper limit given in the UK National Annex is $\mathbf{0.045}\boldsymbol{f_b}$.

Where: f_b is the normalised mean compressive strength of the masonry units for the direction of loadings as obtained by test in accordance with BS EN 772-1[54] or from the manufacturer.

The National Annex has adopted the f_{vk0} values in Code Table 3.4 as an evaluation of a database of test results. Table 4.6 in this *Manual* gives the initial shear strength values in the UK National Annex[4]. In shell bedded masonry, the factor 0.5 in the second formula for f_{vk} (in this section) is replaced by the ratio of the total width of mortar strips (each a minimum of 30mm) to the thickness of the wall. Again, the UK National Annex to EC6 Part 1-1[1] adopts an upper limit of $\mathbf{0.045}\boldsymbol{f_b}$ for the characteristic shear strength.

Table 4.5 Values of α and β to be used for, and limitations on the use of, equation for f_k

Equation showing values of α and β for the limitations given	Limitations
$f_k = K f_b^{0.7} f_m^{0.3}$	For masonry made with general purpose mortar and lightweight mortar.
$f_k = K f_b^{0.85}$	For masonry made with thin layer mortar, in bed joints of thickness 0.5mm to 3mm, and clay units of Group 1 and 4, calcium silicate, aggregate units and autoclaved aerated concrete units.
$f_k = K f_b^{0.7}$	For masonry units made with thin layer mortar, in bed joints of thickness 0.5mm to 3mm, and clay units of Group 2 and 3.

Further limitations

The above three equations can be used provided the following requirements are satisfied:

- the masonry is detailed in accordance with Section 8 of BS EN 1996-1-1[1]
- bed joints and perpend joints made with general purpose and lightweight mortars should have a thickness not less than 6mm nor more than 15mm, and bed and perpend joints made with thin layer mortars should have a thickness not less than 0.5mm nor more than 3mm
- f_b is not taken to be greater than:
  - **110N/mm²** when units are laid in general purpose mortar
  - **50N/mm²** when units are laid in thin layer mortar

  where f_b is derived from BS EN 772-1[54] when the load is applied in the normal orientation, i.e. perpendicular to the normal bed face. Note that f_b is the normalised strength of a unit; if concrete blocks are to be laid flat, then the normalised strength is still used for the design, even if that strength was obtained by testing blocks in the upright position
- f_m is not taken to be greater than 2 f_b nor greater than:
  - **12N/mm²** when units are laid in general purpose mortar
  - **10N/mm²** when units are laid in lightweight mortar
- the coefficient of variation of the strength of the masonry units is not more than 25%.

Table 4.5 Continued

Notes

a This table is based on data from NA[4] to EC6 Part 1-1.

b 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 4.4.

c 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, or for collar jointed walls with or without mortar in the collar joint, the value of K obtained from Table 4.4 is multiplied by 0.8.

d For masonry made of general purpose mortar where Group 2 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.

e Where action effects are parallel to the direction of the bed joints, the characteristic compressive strength may be determined from the above equations with f_b, derived from BS EN 772-1[54], 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[54] taken to be no greater than 1.0. For Group 2 units, K should then be multiplied by 0.5.

f When the perpend joints are unfilled, the above equations may be used, with consideration of any horizontal actions that might be applied to, or be transmitted by, the masonry. See also Clause 3.6.2(4) of EC6 Part 1-1[1].

Table 4.6 Values of the characteristic initial shear strength of masonry f_{vk0}

<table>
<tr><th rowspan="3">Masonry units</th><th colspan="4">f_{vk0}
(N/mm²)</th></tr>
<tr><th colspan="2">General purpose mortar of the Strength Class given</th><th>Thin layer mortar (bed joint ⩾ 0.5mm and ⩽3mm)</th><th>Lightweight mortar</th></tr>
<tr><td>M12</td><td>0.30</td><td rowspan="3">0.30</td><td rowspan="3">0.15</td></tr>
</table>

<table>
<tr><th>Masonry units</th><th colspan="2">General purpose mortar of the Strength Class given</th><th>Thin layer mortar (bed joint ⩾ 0.5mm and ⩽3mm)</th><th>Lightweight mortar</th></tr>
<tr><td rowspan="3">Clay</td><td>M12</td><td>0.30</td><td rowspan="3">0.30</td><td rowspan="3">0.15</td></tr>
<tr><td>M4 and M6</td><td>0.20</td></tr>
<tr><td>M2</td><td>0.10</td></tr>
<tr><td rowspan="3">Calcium silicate</td><td>M12</td><td>0.20</td><td rowspan="3">0.40</td><td rowspan="3">0.15</td></tr>
<tr><td>M4 and M6</td><td>0.15</td></tr>
<tr><td>M2</td><td>0.10</td></tr>
<tr><td>Aggregate concrete</td><td>M12</td><td>0.20</td><td rowspan="3">0.30</td><td rowspan="3">0.15</td></tr>
<tr><td>Autoclaved aerated concrete</td><td>M4 and M6</td><td>0.15</td></tr>
<tr><td>Manufactured stone and dimensioned natural stone</td><td>M2</td><td>0.10</td></tr>
</table>

Note

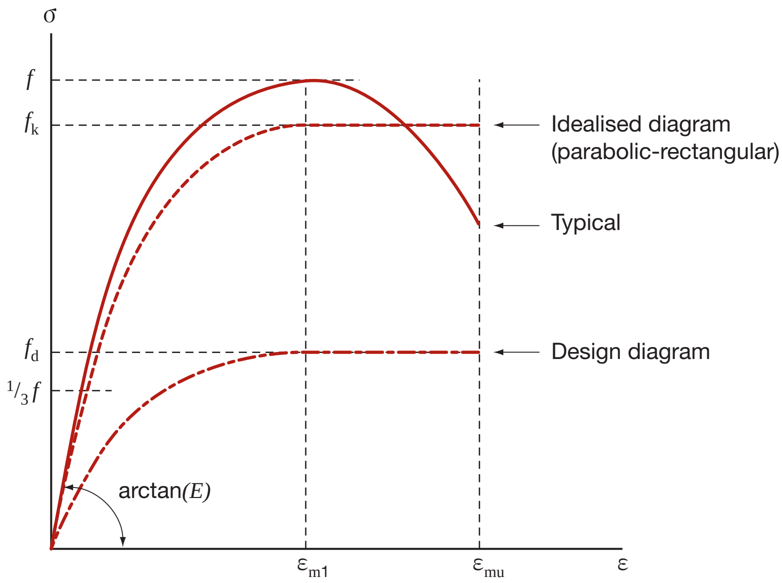
This table is based on data from the NA[4] to EC6 Part 1-1.

4.3.4 Characteristic flexural strength of masonry

The characteristic flexural strength of masonry having a plane of failure parallel to the bed joints f_{xk1} and having a plane of failure perpendicular to the bed joints f_{xk2} are given in the UK National Annex to EC6 Part 1-1[4]. The values of f_{xk1} and f_{xk2} in the UK National Annex for general purpose mortars and clay, calcium silicate and concrete units are given in Table 4.7. For thin layer mortars use the values given for M12 mortar. For lightweight mortars use the values given for M2 mortar.

4.4 Deformation properties

It is rarely necessary to determine the deformation of plain masonry elements. When it is necessary to do so (e.g. laterally loaded panels when outside simple rules), the stress-strain relationship may be taken as linear, parabolic, parabolic rectangular, see Figure 4.1, or as rectangular.



Notes

a This figure is an approximation and may not be suitable for all types of masonry units.

b This figure is based on data from EC6 Part 1.1[1].

Fig 4.1 Stress-strain relationship for masonry in compression

Table 4.7 Characteristic flexural strength of masonry, f_{xk1} and f_{xk2}, in N/mm²

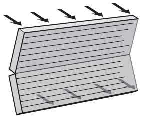
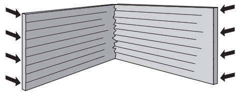
Mortar strength class:	
Clay masonry units of groups 1 and 2 having a declared water absorption[a] of:	
less than 7%	
between 7% and 12%	
over 12%	
Calcium silicate brick sized[b] masonry units	
Aggregate concrete brick sized[b] masonry units	
Aggregate concrete masonry units and manufactured stone of groups 1 and 2 and AAC masonry units, used in walls of thickness up to 100mm[c,d], of declared compressive strength:	
2.9	
3.6	
7.3	
Aggregate concrete masonry units and manufactured stone of groups 1 and 2 and AAC masonry units, used in walls of thickness of 250mm[c,d], or greater, of declared compressive strength:	
2.9	
3.6	
7.3	
Aggregate concrete blocks of groups 1 and 2 and AAC blocks, used in walls of any thickness[c], of declared compressive strength:	
10.4	
⩾17.5	

Notes

a Tests to determine the water absorption of clay masonry units are to be carried out in accordance with BS EN 772-7[55].

b Units not exceeding 337.5mm x 112.5mm.

c 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.

	Values of f_{xk1} – plane of failure parallel to bed joints			Values of f_{xk2} – plane of failure perpendicular to bed joints		
						
	M12	**M6 and M4**	**M2**	**M12**	**M6 and M4**	**M 2**
	0.70 0.50 0.40	0.50 0.40 0.30	0.40 0.35 0.25	2.00 1.50 1.10	1.50 1.10 0.90	1.20 1.00 0.80
	0.30		0.20	0.90		0.60
	0.30		0.20	0.90		0.60
	0.25		0.20	0.40 0.45 0.60		0.40 0.40 0.50
	0.15		0.10	0.25 0.25 0.35		0.20 0.20 0.30
	0.25		0.20	0.75 0.90[e]		0.60 0.70[e]

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

e When used with flexural strength in the parallel direction, assume the orthogonal ratio μ = 0.3.

f This table is based on data from the NA[4] to EC6 Part 1-1.

The short term secant modulus of elasticity E in the UK National Annex[4] to EC6 Part 1-1, is the same as that recommended in the Code, being $1000f_k$.

The long term modulus, based on the short term modulus and allowing for creep, is given by:

$$E_{\text{long term}} = \frac{E}{1 + \phi_\infty}$$

where: E is the short term modulus

ϕ_∞ is the final creep coefficient.

A range of values for the final creep coefficient, the long term moisture expansion or shrinkage coefficients and thermal expansion coefficients are given in EC6 and the specific values in the UK National Annex[4] to EC6 Part 1-1 are given in Table 4.8.

Table 4.8 Values for the final creep coefficient, long term moisture expansion or shrinkage, and coefficients 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

Notes

a The final creep coefficient $\phi_\infty = \varepsilon_{c\infty} / \varepsilon_{el}$ where $\varepsilon_{c\infty}$ is the final creep strain and $\varepsilon_{el} = \sigma/E$.

b Where the long term value of moisture expansion or shrinkage is shown as a negative number it indicates shortening and as a positive number it indicates expansion.

c This table is based on data from the NA[4] to EC6 Part 1-1.

4.5 Design strength

The design value for a material property in the ultimate limit state is obtained by dividing its characteristic value by the appropriate partial factor for materials γ_M. The values in the UK National Annex[4] to EC6 Part 1-1 for γ_M are given in Table 4.9. The execution control classes (quality of workmanship) used in the UK National Annex correspond to the special and normal categories of construction control used in BS 5628[7].

Table 4.9 Values of γ_M for ultimate limit states

Material	γ_M Class of execution control[a,b] 1	2
Masonry		
When in a state of direct or flexural compression		
Unreinforced masonry made with:		
units of category I[c]	2.3[d]	2.7[d]
units of category II[c]	2.6[d]	3.0[d]
When in a state of flexural tension		
units of category I and II[c]	2.3[d]	2.7[d]
When in a state of shear		
Unreinforced masonry made with:		
units of category I and II[c]	2.5[d]	2.5[d]
Steel and other components		
Ancillary components - wall ties	3.5[d]	3.5[d]
Ancillary components - straps	1.5[e]	1.5[e]
Lintels in accordance with BS EN 845-2[34]	See NA to BS EN 845-2[34]	See NA to BS EN 845-2[34]

Notes

a Class 2 of execution control should be assumed whenever the work is carried out following the recommendations of workmanship in BS EN 1996-2[3], including appropriate supervision and inspection.
Class 1 of execution control may be assumed where the criteria for Class 2 of execution control are met 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[1];
- the mortar conforms with BS EN 998-2[31], 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[56] and BS EN 1015-11[57], indicate conformity to the strength requirements given in BS EN 1996-1-1[1] and regular testing of the mortar used on site, in accordance with BS EN 1015-2[56] and BS EN 1015-11[57], should show that the strength requirements of BS EN 1996-1-1[1] are being maintained.

b Masonry wall panels reinforced with bed joint reinforcement used:
- to enhance the lateral strength of the masonry panel or
- 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.

c Category of manufacturing control of masonry unit, see Section 3.3.

d When considering the effects of misuse or accident these values may be halved.

e For horizontal restraint straps, unless otherwise specified, the declared ultimate load capacity depends on a design compressive stress in the masonry of at least 0.4N/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 factor of 3 should be used.

f This table is based on data from NA[4] to EC6 Part 1-1.

Although detailed calculations are not generally required for the serviceability limit state, where they are necessary, the UK National Annex value to be used for γ_M is **1.0**.

In arriving at the class of execution control to be used in design, the following factors should be considered:

- the level and quality of supervision provided by the contractor
- the level and quality of inspectors independent of the contractor. In Design and Build contracts the designer may be considered as an independent inspector where appropriate
- assessment of the site properties of the mortar and concrete infill
- methods of batching and mixing of the constituents of the mortar.

4.6 Accidental damage

EC6[1] gives limited guidance for dealing with accidental damage. It requires that design ensures that there is a reasonable probability that damage resulting from misuse or accident will not be disproportionate to the cause of that damage. The designer must consider the structural behaviour in accidental situations accordingly, using one of the following methods:

i) members designed to resist the effects of accidental actions given in BS EN 1991-1-7[47]

ii) the hypothetical removal of essential loadbearing members in turn

iii) the provision of a tying system

iv) the provision of measures to reduce the risk of accidental actions occurring, e.g. provision of impact barriers.

Methods ii) and iii) follow current UK practice for which guidance is given in a BDA/CBA/AACPA publication[58] and Approved Document A to the Building Regulations[10]. Guidance from BS 5628 Part 1[7] is included in Appendix B to this *Manual*.

5.1 Combination of actions

Combinations of actions (load combinations) for unreinforced masonry design to EC6 Part 1-1[1] are to be based on the rules given in BS EN 1990[51] *Basis of structural design.* For the majority of normal residential and office structures simplified combinations may be used. Thus the variable actions (imposed loads) may be treated as one action, i.e. taken as equal loading on all spans or zero as appropriate. Permanent actions (dead loads) are taken as the same on each span, the value of the partial factor used depending upon whether the action is favourable or unfavourable (see also Section 4.2.1). The combinations to be considered for normal masonry buildings become those given in Table 5.1 which also gives the partial factors to be applied.

5.2 Design procedure

A suitable procedure for the design of loadbearing masonry structures is given below.

i) Consider overall stability. Check that lateral forces are resisted by arranging suitable shear walls, positioned as symmetrically as possible. Check that floors and roof can act as horizontal diaphragms to transfer lateral loads to shear walls. Consider the need for movement joints through the structure and the stability of the independent structural sections formed by these joints. Consider whether allowance for sway of the structure is necessary (see Section 5.3.1).

ii) Consider robustness (see Sections 2.7 and 4.6).

iii) Determine minimum requirements of masonry unit quality and mortar strength for durability (see Chapter 3).

iv) Determine requirements for minimum thickness of members for fire resistance (see Chapter 7).

v) Check that the requirements for such matters as thermal value, sound transmission, aesthetics, durability, dpcs and partitions have been met.

vi) Select worst case loading situations for design (e.g. most heavily loaded; minimum vertical load and maximum lateral wind load; wind uplift possibly inducing tension), checking the points of lateral support and any anchorage assumed in the calculations can be achieved in practice. Consider the effects of any damp proof courses (particularly in narrow piers between windows), services penetrations of the walls and joints.

vii) Make calculations.

viii) Prepare details and specifications, and include provision for movement both of individual masonry elements and of overall building.

5.3 Walls and piers subject to vertical loading

5.3.1 General

The design of vertical loadbearing masonry elements is based on consideration of slenderness and buckling. The edge restraint conditions of the masonry elements are therefore critical to their design. Edge restraint is provided in the form of either vertical or horizontal lateral support to the edges of masonry walls. These lateral supports (e.g. buttressing or stiffening walls, floors or roofs acting as horizontal diaphragms) must be capable of transmitting the appropriate lateral forces to the strongpoints in the structure (shear walls etc.). EC6 Part 1-1[1] has no specific requirement for these lateral loads. In the UK, it is usual practice to design the lateral support for the sum of:

i) the simple static reactions to the total applied design horizontal forces (e.g. wind loads) at the lateral support, and

ii) 2.5% of the total design vertical load that the wall or column is designed to carry applied as a horizontal force at the line of lateral support.

The resulting horizontal force is applied to all individual horizontal and vertical lateral supports and their connections. It is not to be confused with the global lateral load which is used for the overall stability of design of the structure and is distributed between the strongpoints (shear walls) in the structure (see Section 4.1).

In certain circumstances, however, it may be appropriate to design the strongpoints to resist the sum of these forces, e.g. ground-storey of podium construction, two-storey building and long-span heavily-loaded first floor.

As well as the vertical loads applied directly to a wall or pier, consideration must be given to eccentricity of load resulting from the layout of the masonry elements, the arrangement of the floor to masonry junctions and from construction deviations and differences in the material properties of individual components.

Table 5.1 Combinations of actions with values of γ_G γ_Q ψ_0

Combinations of actions	Load Type				
	Permanent actions G_k		Leading variable action Q_{k1}		Additional variable actions Q_{ki}
	Unfavourable $\gamma_{G,sup}$	Favourable $\gamma_{G,inf}$	Unfavourable γ_Q	Favourable γ_Q	γ_{Qi} ψ_0
Permanent plus variable (leading only)	1.35	1.0	1.5	0	–
Permanent plus leading variable (e.g. wind)	1.35	1.0	1.5	–	–
Permanent plus leading variable plus additional variable (e.g. wind)	1.35	1.0	1.5	–	1.5 ψ_0[a]

Note

a Value of ψ_o from Table 4.2 (e.g. 1.5 x 0.5).

Structures which rely on the bending resistance of the masonry for their overall stability are specifically excluded from the scope of this *Manual* (see Section 1.3). Thus masonry shear walls (or other bracing) are to be provided to prevent sway of the structure. The adequacy of the shear walls or bracing can be checked in accordance with equation 5.1 in EC6 Part 1-1[1] as below.

Allowance for sway is unnecessary if:

$$h_{tot}\sqrt{\frac{N_{Ed}}{\sum EI}} \quad \begin{aligned} &\leqslant 0.6 \text{ for } n \geqslant 4 \\ &\leqslant 0.2 + 0.1n \text{ for } 1 \leqslant n \leqslant 4 \end{aligned}$$

Where: 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 top of the foundation)

$\sum EI$ is the sum of the stiffnesses of all vertical stiffening building elements (shear walls) in the relevant direction

n is the number of storeys.

Openings of less than 2m^2 in area, in a shear wall, with a height not exceeding 0.6 × the clear height of the wall, can be neglected when considering the stiffness of the wall.

If this relationship cannot be satisfied initially, the most practical solution is to increase the provision of shear walls until it can be satisfied.

5.3.2 Effective height of walls

The effective height of a loadbearing wall is assessed by allowing for the relative stiffness of the elements of structure (floors, roof, walls etc.) connected to the wall and the efficiency of the connections.

The effective height $h_{ef} = \rho_n h$

Where: h is the clear storey height of the wall

ρ_n is a reduction factor depending upon the edge restraint or stiffening of the wall. n may be 2, 3 or 4 depending upon the form of restraint (number of restrained edges).

The value of ρ_n to be used in various situations is illustrated in Figure 5.1.

Walls may be considered to be stiffened along a vertical edge if a suitable masonry stiffening wall or other stiffening element (e.g. r.c. column) is adequately connected along that edge. Figures 5.2 and 5.3 illustrate the requirements for a masonry stiffening wall.

Walls stiffened on two vertical edges, with $l \geqslant 30t$ or walls stiffened on one vertical edge with $l \geqslant 15t$, should be treated as restrained top and bottom only, see Figure 5.4. Whilst EC6 uses 't' it seems reasonable to use t_{ef} for cavity walls.

The effect of vertical chases and/or recesses should be allowed for (see Section 6.4). A free edge should be assumed if the thickness of the wall remaining is less than half the wall thickness.

A free edge should also be assumed at the edge of any opening in the stiffened wall which is greater than 0.25h in height or 0.25l in length or has an area greater than 10% of the area of the stiffened wall.

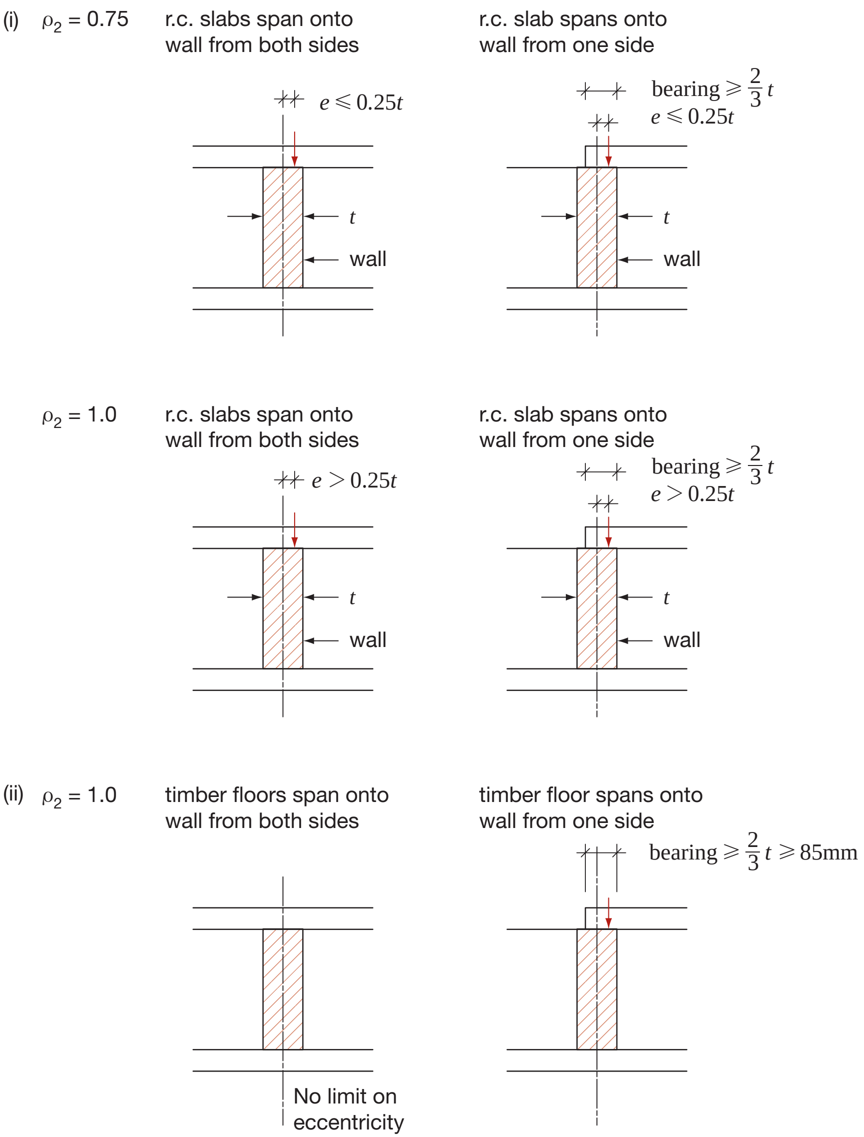
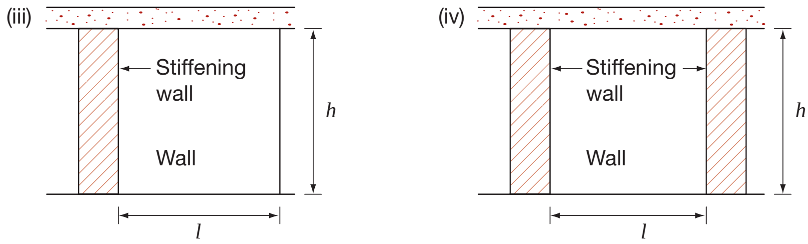


Fig 5.1a Values of effective height reduction factor, ρ_n



$$\rho_3 = \frac{1}{1 + \left[\frac{\rho_2 h}{3l}\right]^2} \rho_2$$

Where ρ_2 is obtained from (i) or (ii) from Fig 5.1a, as appropriate, if $h \leqslant 3.5l$,

or if $h > 3.5l$

$$\rho_3 = \frac{1.5l}{h} \geqslant 0.3$$

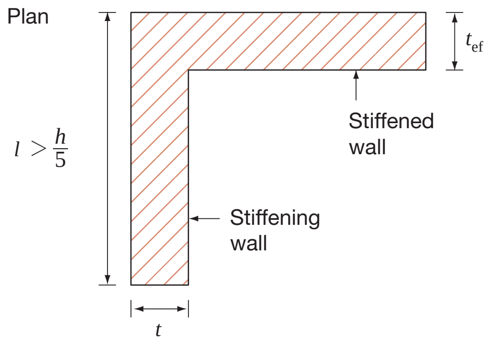
$$\rho_4 = \frac{1}{1 + \left[\frac{\rho_2 h}{l}\right]^2} \rho_2$$

Where ρ_2 is obtained from (i) or (ii) from Fig 5.1a, as appropriate, if $h \leqslant 1.15l$,

or if $h > 1.15l$

$$\rho_4 = \frac{0.5l}{h}$$

Fig 5.1b Values of effective height reduction factor, ρ_n



Notes

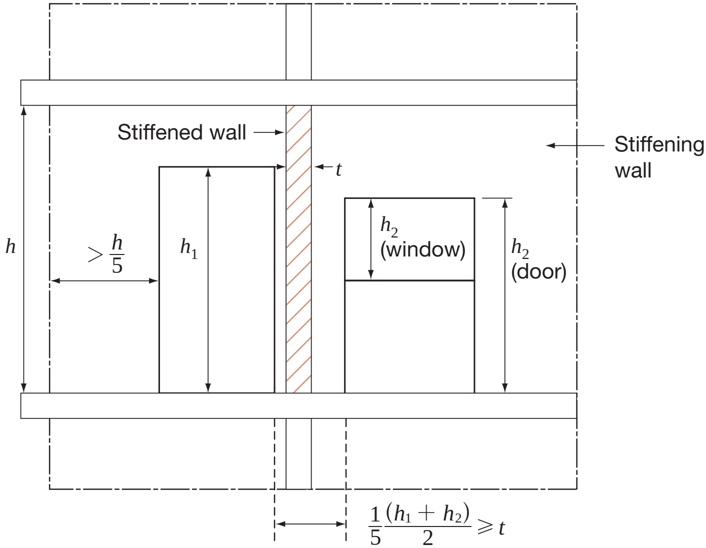
a $t \geqslant 0.3\ t_{ef}$

b t_{ef} is the effective thickness of the stiffened wall.

c h is the clear height of the stiffened wall.

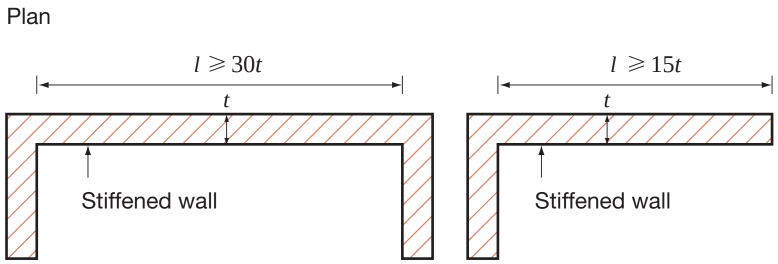
d Junction between walls must be unlikely to crack, e.g. loads on walls and materials etc. should be similar, junction should be fully bonded or adequately tied together to resist tension and compression forces.

Fig 5.2 Requirements for stiffening wall



Note
This figure is based on data from EC6 Part 1.1[1].

Fig 5.3 Minimum length of stiffening wall with openings



Notes

a These walls to be treated as restrained top and bottom only, i.e. ignore effect at vertical edge stiffening.

b t may be taken as t_{ef} for cavity walls.

Fig 5.4 Limitations on lengths of stiffened walls

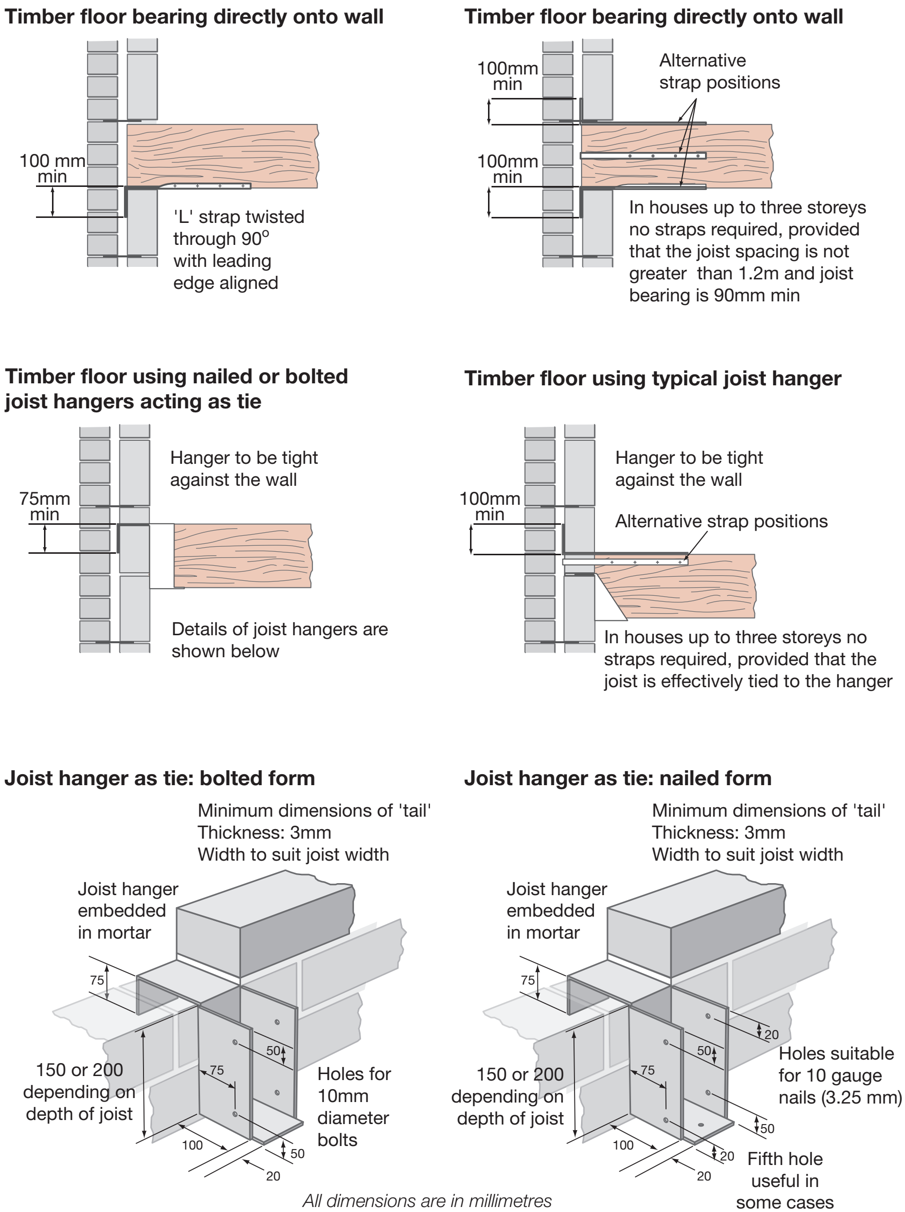
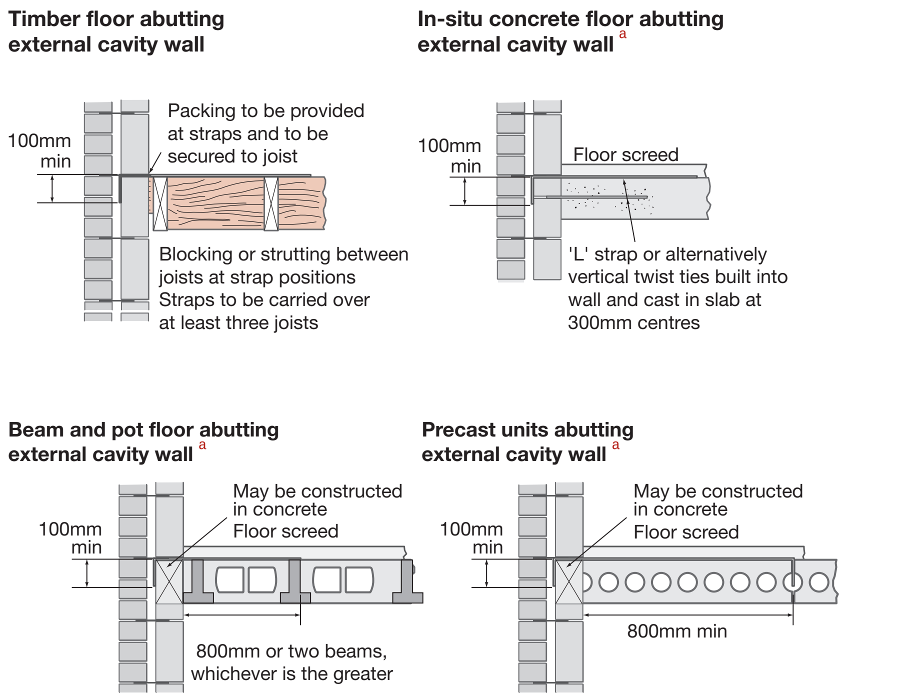


Fig 5.5 Connections suitable for timber floor joists spanning onto masonry walls from one side only

As can be seen from Figure 5.1, a simplistic approach to horizontal lateral support is adopted in EC6 Part 1-1[1] , particularly with respect to timber floor construction. In the UK, it is considered good practice to ensure that timber floors are well fixed to their supporting walls. Suitable connections are illustrated in Figures 5.5 to 5.8.

UK practice also permits the assumption of lateral support to a masonry wall in buildings of not more than 3 storeys, where timber floor joists, spaced not more than 1.2m apart, are supported by the wall on suitable hangers effectively fixed to the joists, see Figure 5.5. For higher buildings and where floors abut a masonry wall, lateral support may be assumed, provided that the floor structure is adequately connected to the wall. Suitable details are illustrated in Figures 5.6 and 5.7. In these situations, it would be appropriate to use a ρ_n value of 1.0. Although the figures show floor details, they also apply to roofs, provided that equivalent connections are used.

Figure 5.8 illustrates UK practice for the provision of lateral support to gable walls using truss roof construction, again ρ_n should be taken as 1.0.



Note

a Deflection of floor at junction with wall could cause screed to crack at strap locations.

Fig 5.6 Connections suitable for floors which span parallel to masonry walls (one side only)

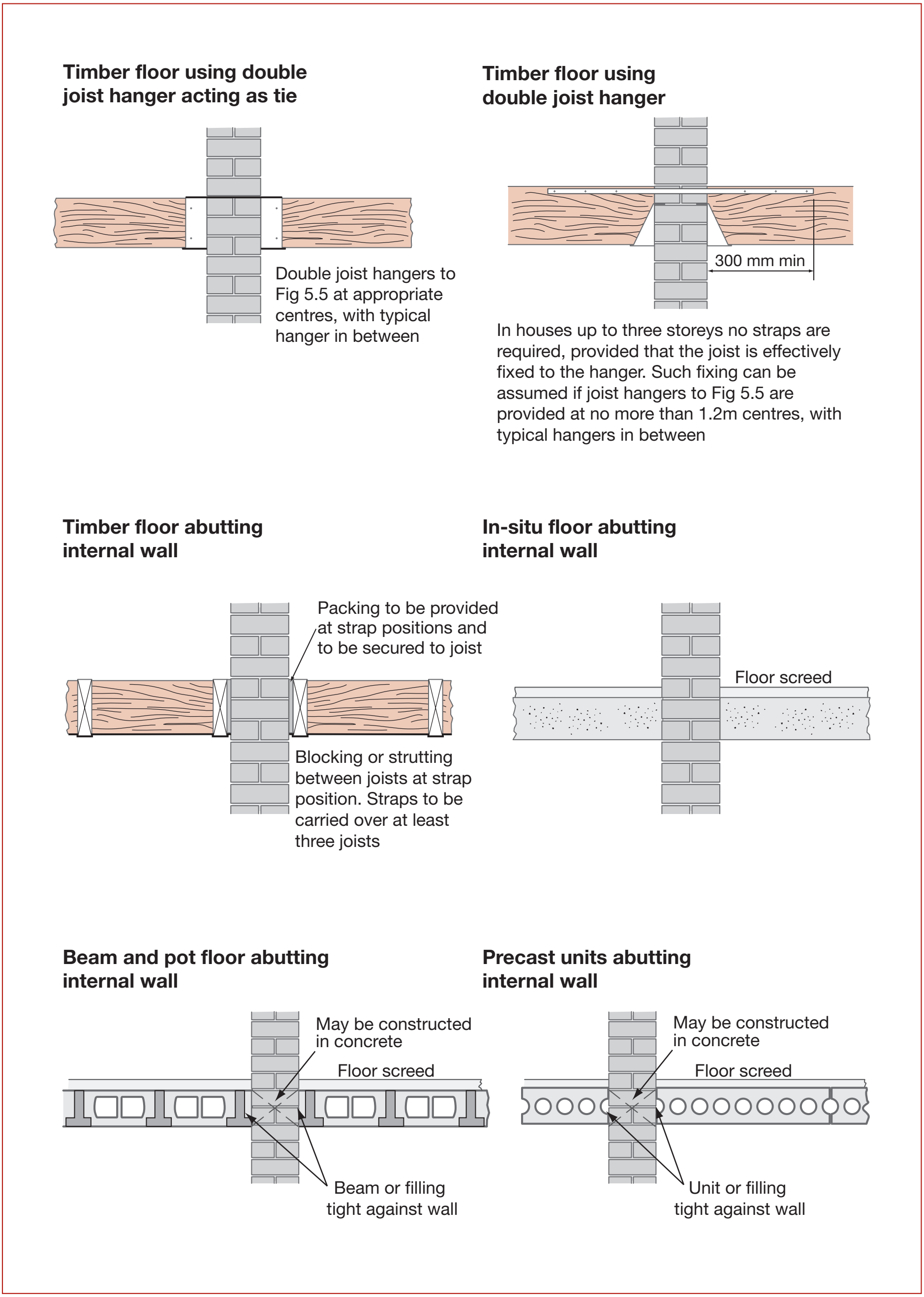
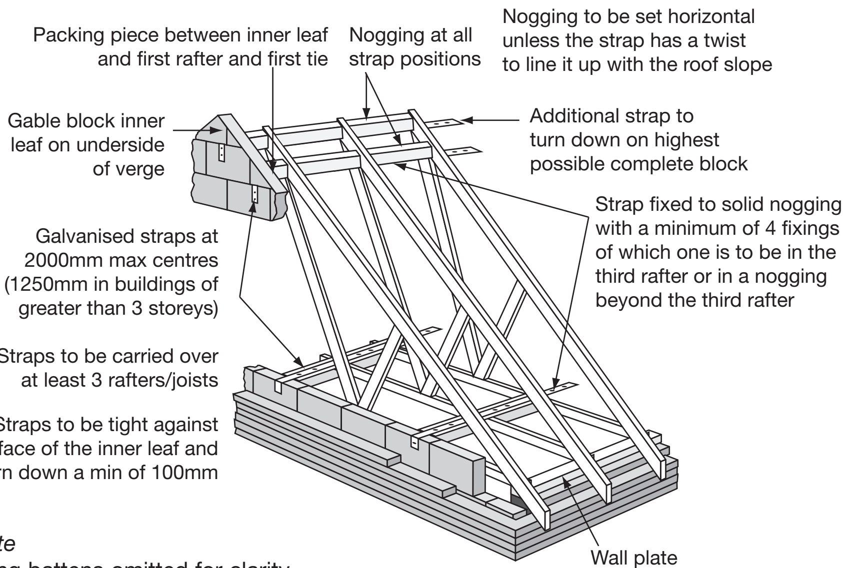


Fig 5.7 Connections suitable for floors to internal masonry walls

Truss roofs with straps



Note
Tiling battens omitted for clarity.

Fig 5.8 Connections suitable for truss roofs to masonry gable walls

5.3.3 Effective length

The concept of effective length is not used in EC6 Part 1-1[1] but the effect of vertical edge restraint is to modify the effective height (see Section 5.3.2).

5.3.4 Effective thickness

The effective thickness t_{ef} of a single-leaf wall, a double-leaf (collar jointed) wall, a faced wall and a shell bedded wall is equal to the actual thickness of the wall t.

In an unreinforced grouted cavity wall (it may have bed joint reinforcement), t_{ef} is taken as the overall wall thickness, including the filled cavity. The width of the cavity should be taken as its actual width but should not be taken as greater than 100mm.

The effective thickness of a wall stiffened by piers is obtained from:

$$t_{ef} = \rho_t t$$

Where: t_{ef} is the effective thickness
ρ_t is a coefficient (see Table 5.2)
t is the thickness of the wall.

For a cavity wall $t_{ef} = \sqrt[3]{t_1^3 + t_2^3}$ using the UK National Annex[4] value of $k_{tef} = \mathbf{1}$ (see below).

Where: t_1, t_2 are the actual or effective thicknesses of the two leaves as appropriate. When relevant, t_1 is the thickness of the outer, unloaded leaf, and t_2 is the thickness of the inner or loaded leaf.

Note that EC6 gives the effective thickness of a cavity wall as:

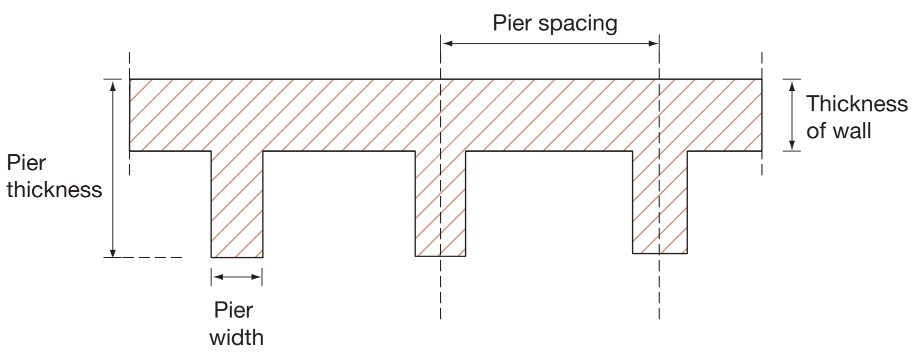
$$t_{ef} = \sqrt[3]{k_{tef}\, t_1^3 + t_2^3}$$

Where: k_{tef} ($\leqslant 2$) is a factor to allow for the relative E values of the leaves and is given in the UK National Annex[4] as **1.0**.

Table 5.2 Stiffness coefficient ρ_t, for walls stiffened by piers

Ratio of pier spacing (centre to centre) to pier width	Ratio of pier thickness to actual thickness of wall to which it is bonded		
	1	**2**	**3**
6	1.0	1.4	2.0
10	1.0	1.2	1.4
20	1.0	1.0	1.0

Note
Linear interpolation between the values given in this table is permissible.



In a cavity wall, when calculating its effective thickness, the thickness of the unloaded leaf may not be taken as greater than the loaded leaf, and the two leaves must be tied together in accordance with the requirements of Clause 6.5 in EC6 Part 1-1[1]. Generally, the minimum spacing of wall ties (**2.5/m²**) given in the UK National Annex[4] will govern the provision of wall ties. This accords with the guidance in Annex C of BS 5628 Part 1[7].

In some cases it may be necessary to check the minimum requirement by calculation. EC6 requires that the minimum number of wall ties n_t should be obtained from:

$$n_t \geqslant \frac{W_{Ed}}{F_d}$$

Where: W_{Ed} is the design value of the horizontal load, per unit area, to be transferred

F_d is the design compressive or tensile resistance of a wall tie, as appropriate to the design condition.

When determining the effective thickness of a wall stiffened by intersecting walls, the walls may be considered as piers of equal thickness and t_{ef} obtained accordingly as above. The pier depth should be taken as three times the thickness of the stiffened wall.

Although the effective thickness of a diaphragm wall can be shown to be greater than the actual overall thickness of the diaphragm wall, in design it is usual to take the effective thickness as its actual thickness.

5.3.5 Slenderness ratio

The slenderness ratio is given by:

$$\frac{h_{ef}}{t_{ef}}$$

It should not be greater than 27 when subjected to mainly vertical loading.

5.3.6 Eccentricity at right angles to the wall

5.3.6.1 General

The design of vertically loaded masonry elements is based on the slenderness of the element and the eccentricity of the applied vertical load. EC6 Part 1-1[1] gives formulae for calculating the eccentricity to be used in design based on the model shown in Figure 5.9. This is a significant change in approach from that used in BS 5628[7].

EC6 requires the verification of the resistance of a vertically loaded wall at the top, the bottom and the mid height.

5.3.6.2 Eccentricity at the top and bottom of the wall

The eccentricity at the top and bottom of the wall e_i used in design includes allowance for an initial eccentricity e_{init} (for construction imperfections) and e_{he} the eccentricity at top or bottom of wall resulting from lateral, e.g. wind loading, as well as that arising from floor loads etc.

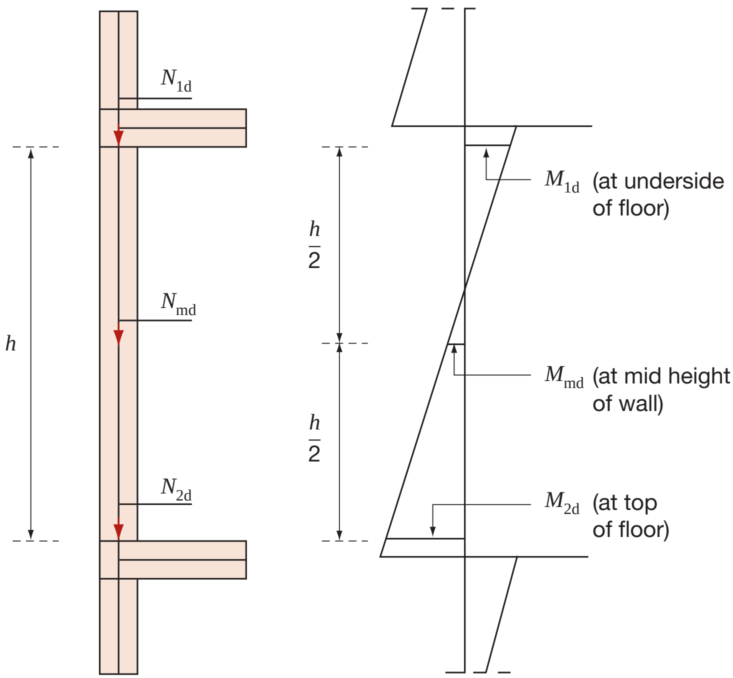
Thus $e_i = \frac{M_{id}}{N_{id}} + e_{he} \pm e_{init}$

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

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

e_{init} may be taken as h_{ef} /450.

If e_i is equal to or less than 0.05t the reduction factor ϕ for load capacity is taken as 0.9.



Note
This figure is based on data from EC6 Part 1.1[1].

Fig 5.9 Moments from calculation of eccentricities

5.3.6.3 Eccentricity at the mid height of the wall

The eccentricity at the mid height of the wall e_{mk} used in design includes allowance for initial eccentricity e_{init}, eccentricity due to creep e_k, lateral load eccentricity e_{hm}, and load eccentricity e_m.

The UK National Annex[5] states that e_k may be ignored if the slenderness ratio of the wall is not greater than **27.** Thus, effectively there is never any need to allow for eccentricity due to creep in vertical load design.

The mid height eccentricity due to loads is given by:

$$e_{mk} = \frac{M_{md}}{N_{md}} + e_{hm} \pm e_{init}$$

Where: M_{md} is the design value of the greatest moment at mid height of the wall resulting from the moments at the top and bottom of the wall (see Figure 5.9), 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 mid height of the wall, including any load applied eccentrically to the face of the wall (e.g. brackets).

e_{hm} need only be used in the appropriate load combination and its sign relative to the load eccentrically should be considered.

Again, e_{init} may be taken as $h_{ef}/450$.

Because eccentricity due to creep can be ignored, $e_{mk} = e_m$. Again if e_{mk} is not greater than $0.05t$ the reduction factor Φ for load capacity is taken as 0.9.

5.3.6.4 Simplified sub-frame analysis

Annex C to EC6 Part 1-1[1] gives a simplified method for obtaining the moments at the top and bottom of vertically loaded walls. This is based on a sub-frame analysis as shown in Figure 5.10. The ends of the members remote from the junction should be taken as fixed unless they are known to take no moment at all, when they may be taken to be hinged. The end moment at node 1, M_1 may be calculated from the equation below and the end moment at node 2, M_2 similarly but using $n_2E_2I_2/h_2$ instead of $n_1E_1I_1/h_1$ in the numerator.

$$M_1 = \frac{\frac{n_1E_1I_1}{h_1}}{\frac{n_1E_1I_1}{h_1} + \frac{n_2E_2I_2}{h_2} + \frac{n_3E_3I_3}{l_3} + \frac{n_4E_4I_4}{l_4}} \left[\frac{w_3 l_3^2}{4(n_3 - 1)} - \frac{w_4 l_4^2}{4(n_4 - 1)}\right]$$

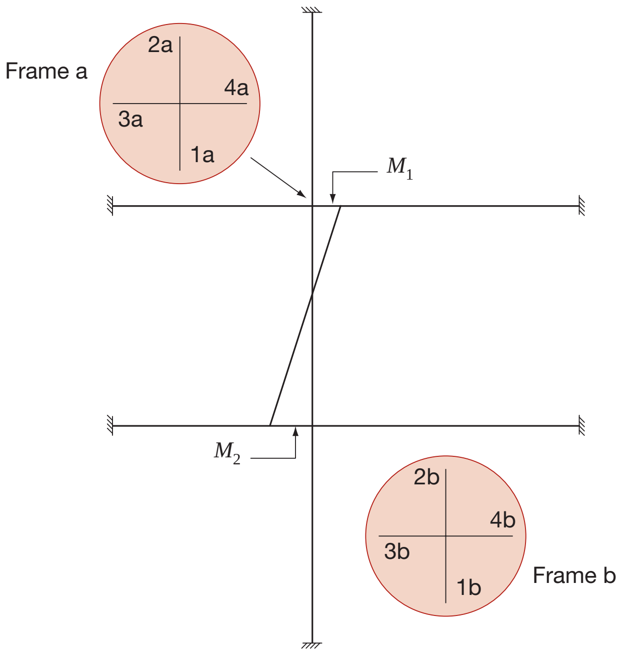
Where: n_i is the stiffness factor of member i, where i = 1, 2, 3, or 4, and is taken as 4 for members fixed at both ends and otherwise 3

E_i is the modulus of elasticity of member i, where i = 1, 2, 3 or 4; (Note: It will normally be sufficient to take values of E_i as 1000 f_k for all masonry units)

I_i is the second moment of area of member i, where i = 1, 2, 3 or 4 (in case of a cavity wall in which only one leaf is loadbearing, I_i should be taken as that of the loadbearing leaf only)

h_1 is the clear height of member 1
h_2 is the clear height of member 2
l_3 is the clear span of member 3
l_4 is the clear span of member 4
w_3 is the design uniformly distributed load on member 3, using the partial factors from BS EN 1990[51], unfavourable effect
w_4 is the design uniformly distributed load on member 4, using the partial factors from BS EN BS 1990[51], unfavourable effect.

Where column or beam members do not exist, they are simply omitted from the diagram and the end moment equation amended accordingly.



Notes
a Moment M_1 is found from Frame a and Moment M_2 from Frame b.
b This figure is based on data from EC6 Part 1.1[1].

Fig 5.10 Simplified frame diagram

EC6 Part 1-1[1] does not consider the method to be appropriate for use with timber joist floors or where the calculated eccentricity exceeds 0.45 times the wall thickness. In these cases, the eccentricity of loading may be based on the load being resisted by the minimum required bearing depth, provided the depth is not taken as greater than 0.1 times the wall thickness, stressed to the appropriate design strength of the material. Figure 5.11 illustrates the approach.

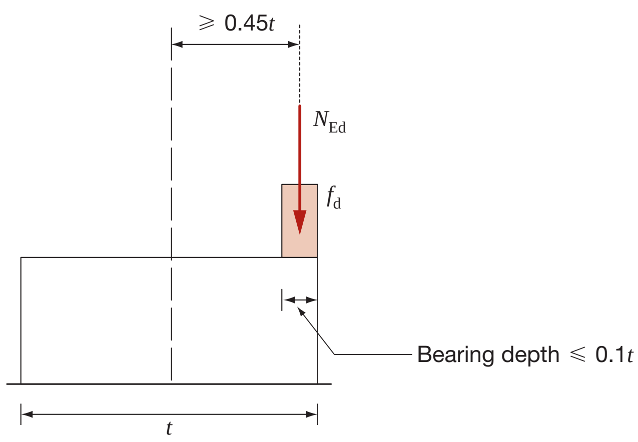
When a floor is supported over part of the thickness of a wall, see Figure 5.12, the moment above the floor M_{Edu} and the moment below the floor M_{Edf} may be obtained from the expressions below, provided that the values are less than are obtained from the sub-frame analysis above:

$$M_{Edu} = N_{Edu} \frac{(t - 3a)}{4}$$

$$M_{Edf} = N_{Edf} \frac{a}{2} + N_{Edu} \frac{(t + a)}{4}$$

Where: N_{Edu} is the design load in the upper wall
N_{Edf} is the design load applied by the floor
a is the distance from the face of the wall to the edge of the floor.

The EC6 approach to bending in walls and columns assumes that they are in double curvature with a point of contraflexure in the height of the element. Whilst this will be the case in the majority of masonry structures, it is possible that this model may not be entirely appropriate (e.g. certain single-storey structures, see sections 5.4.5 and 5.4.7) and the designer may need to adjust the idealised moment diagram accordingly.



Note
This figure is based on data from EC6 Part 1.1[1].

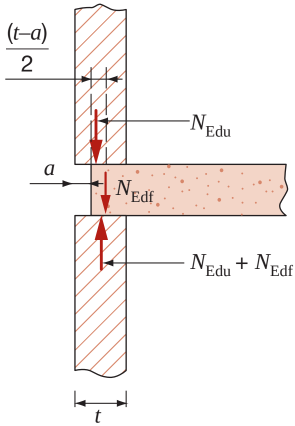
Fig 5.11 Eccentricity obtained from design load resisted by stress block

5.3.6.5 UK Practice

BS 5628[7] practice is to adopt notional eccentricities for floor loadings (the floor loading is considered to act at one-third of the bearing width of floor from the inside face of the supporting wall) and to consider that the load from the wall above the joint is axial. These assumptions enable the eccentricity for derivation of a slenderness and eccentricity reduction factor to be calculated from the relevant formulae (6.5 and 6.7) in the code[1], and may, therefore, be said to comply with the requirement in Clause 5.5.1.1(a) of EC6 Part 1-1[1] to calculate eccentricities from a knowledge of the layout of the walls, the interaction of the floors and the stiffening walls. However, there has never been a thorough going justification for these assumptions, which were based on an approach used in previous Codes of Practice for braced reinforced concrete frames[59]. The methods used in BS 5628[7] and EC6 are, therefore, considered to be incompatible and should not be used in combination.

5.3.7 Vertical load resistance

At the ultimate limit state the design value of the vertical load N_{Ed} shall be less than the design value of the vertical resistance of the wall N_{Rd}.



Note
This figure is based on data from EC6 Part 1.1[1].

Fig 5.12 Diagram showing the forces when a floor is supported over a part of the thickness of a wall

5.3.8 Vertical load resistance of solid walls and columns

It should be noted that there are no separate provisions for columns in EC6 Part 1-1[1]. They are simply considered as short walls.

The design resistance of a single leaf wall per unit length is N_{Rd} given by:

$$N_{Rd} = \Phi_t f_d$$

Where: Φ is a capacity reduction factor allowing for the effects of slenderness and eccentricity of loading

t is the thickness of the wall

f_d is the design compressive strength of the masonry.

When the cross-sectional area of the wall is less than 0.1m², f_d should be multiplied by:

$$(0.7 + 3A)$$

Where: A is the loadbearing horizontal gross cross-sectional area of the wall in square metres.

For a faced wall, the wall may be designed as a single leaf wall constructed entirely of the weaker unit with a longitudinal joint.

For a double-leaf wall, if the leaves are tied together adequately, the wall may be designed as a single leaf wall (assuming that both leaves are similarly loaded), or alternatively as a cavity wall.

Chases and recesses should be allowed for, see Section 6.4.

The slenderness reduction factor Φ is applied at the top or bottom of the wall Φ_i and at mid height of wall, Φ_m.

$$\Phi_i = 1 - 2\frac{e_i}{t}$$

Where: e_i is the eccentricity at the top or bottom of the wall, see Section 5.3.6.2

t is the thickness of the wall.

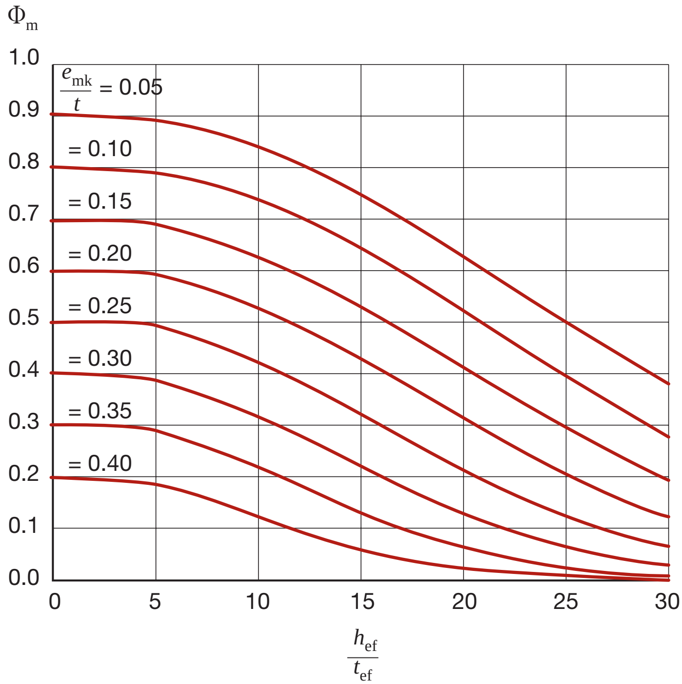
$$\Phi_m = A_1^{e - \frac{u^2}{2}}$$

Where: $A_1 = 1 - 2\frac{e_{mk}}{t}$

$$u = \frac{\lambda - 0.063}{0.73 - 1.17\frac{e_{mk}}{t}}$$

$$\lambda = \frac{h_{ef}}{t_{ef}}\sqrt{\frac{f_k}{E}}$$

As discussed in Section 5.3.6.3, e_{mk} is taken as equal to e_m where e_m is the eccentricity in the mid height of the wall.



Note
This figure is based on data from EC6 Part 1.1[1].

Fig 5.13 Values of Φ_m against slenderness ratio for different eccentricities, based on an E of $1000f_k$

Annex G of EC6 Part 1-1[1] gives a graphical representation of Φ_m for an E value of 1000 f_k which is the value given in the UK National Annex[4]. The graph is reproduced here as Figure 5.13.

5.3.9 Vertical load resistance of cavity walls and columns

For cavity walls and columns (treated as short cavity walls) each leaf should be verified separately using a slenderness ratio based on the effective thickness of the wall.

5.3.10 Eccentricity in the plane of the wall and shear wall

The eccentricity in the plane of a wall can be calculated from statics alone, see Figure 5.14. Where a horizontal force is resisted by several walls it may be distributed between the walls in proportion to their flexural stiffness about an axis perpendicular to the direction of the force. The forces in the walls may be determined by an appropriate elastic analysis. The compressed section of the wall should be verified for the vertical loading including the effects of shear load.

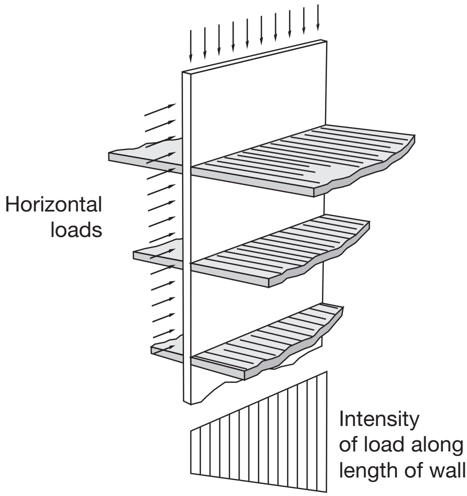


Fig 5.14 Load distribution from loading eccentric to plane of wall

5.3.11 Horizontal shear resistance

At the ultimate limit state the design shear resistance V_{Rd} must be equal to or greater than the design shear load V_{Ed}.

Where: V_{Rd} = $f_{vd}\, t\, l_c$

and f_{vd} is the design value of the shear strength of masonry (see Sections 4.3.3 and 4.5)

t is the thickness of the wall resisting shear or, in the case of a cavity wall, the sum of the thicknesses of the leaves

l_c is the length of the compressed part of the wall, ignoring any part of the wall that is in tension.

l_c should be calculated assuming a linear stress distribution of the compressive stresses, allowing for any openings, chases etc.

5.3.12 Vertical shear resistance

The connections between shear walls and their flanges formed by intersecting walls should be checked for vertical shear. The vertical shear resistance for bond and masonry should be obtained using f_{vk0} (see Section 4.3.3) unless test results or other evidence are available to justify a higher figure. Where metal ties or similar are used to bond the two walls appropriate characteristic shear strengths should be used.

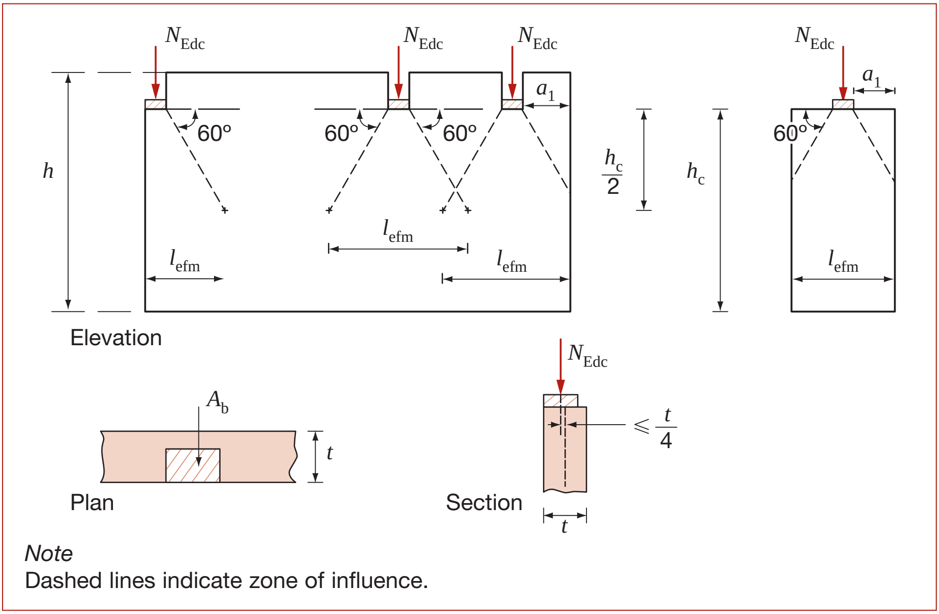


Fig 5.15 Walls subjected to concentrated load

5.3.13 Concentrated loads

Increased local stresses may be permitted beneath the bearing of a concentrated load of a purely local nature, such as beams, columns, lintels, etc. provided that either the element applying the load is sensibly rigid or a suitable spreader is introduced.

The enhancement depends upon the masonry unit group, and its location to the wall. See Figure 5.15.

The vertical concentrated load resistance of the wall N_{Rdc} is given by:

$$N_{Rdc} = \beta A_b f_d$$

Where: $\beta = \left[1 + 0.3\frac{a_1}{h_c}\right]\left[1.5 - 1.1\frac{A_b}{A_{ef}}\right] > 1.0$

β should not be taken as greater than:

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

Where: β is an enhancement factor for concentrated loads

a_1 is the distance from the end of the wall to the nearer edge of the loaded area (see Figure 5.15)

h_c is the height of the wall to the level of the load

A_b is the loaded area

A_{ef}	is the effective area of bearing, i.e. $l_{efm}\, t$
$\frac{A_b}{A_{ef}}$	is not to be taken as greater than 0.45
l_{efm}	is the effective length of the bearing as determined at the mid height ($h_c/2$) of the wall or pier (see Figure 5.15)
t	is the thickness of the wall, taking into account the depth of recesses in joints greater than 5mm.

Note that values for the enhancement factor for β are shown in graphical form in Figure 5.16.

For walls built with other than Group 1 units or when shell bedding is used, a check should be made on the design compressive strength locally under the concentrated load, taking β equal to 1.0. Where the eccentricity of the concentrated load exceeds $t/4$, no enhancement may be taken. Where a suitable spreader (height > 200mm, length > 3 times bearing length, width equal to bearing wall) is provided, the design value of the compressive stress beneath the concentrated load should not exceed $1.5f_d$.

In all cases, the mid height capacity of the wall should be checked allowing for the effects of the concentrated loads, particularly if their zones of influence overlap, see Figure 5.15.

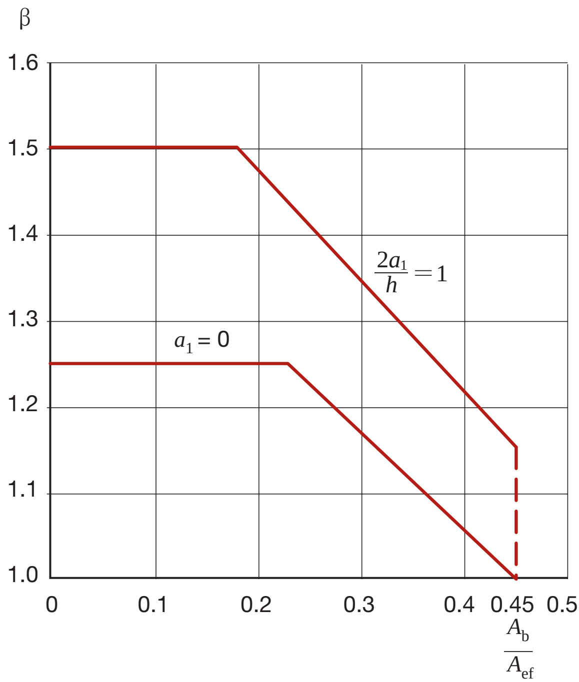


Fig 5.16 Graph showing the enhancement factor β for concentrated loads under bearings

5.4 Walls subject to lateral loading

5.4.1 General

Where walls are subjected to lateral loading (e.g. wind), the walls may be considered as one-way or two-way spanning panels acting in flexure depending upon their edge support conditions. Alternatively, where appropriate, arching action may be relied upon to resist lateral loading. The flexural design approach adopted in EC6 Part 1-1[1] is effectively that used in BS 5628 Part 1[7].

5.4.2 Limits on wall panel sizes

Generally in EC6, design to satisfy the ultimate limit state is also deemed to satisfy the serviceability limit state. However, in the case of the design of laterally loaded wall panels, EC6 provides limiting dimensions for the panels to avoid serviceability problems resulting from possible deflection, creep, shrinkage, temperature effects and cracking movements.

These limiting dimensions are given in the form of graphs and are reproduced here in Figures 5.17, 5.18 and 5.19 where h is the clear height of the wall; l is the length of the wall and t is the thickness of the wall (taken as t_{ef} for cavity walls).

For wall panels supported at the top and bottom only, h should be limited to $30t$. t must not be less than 100mm (one leaf of a cavity wall also) to use these graphs.

It will be noted that the slenderness ratio limit of 27 used in Section 5.3.5 for vertically loaded walls is exceeded in these graphs. This is considered to be acceptable for laterally loaded wall panels.

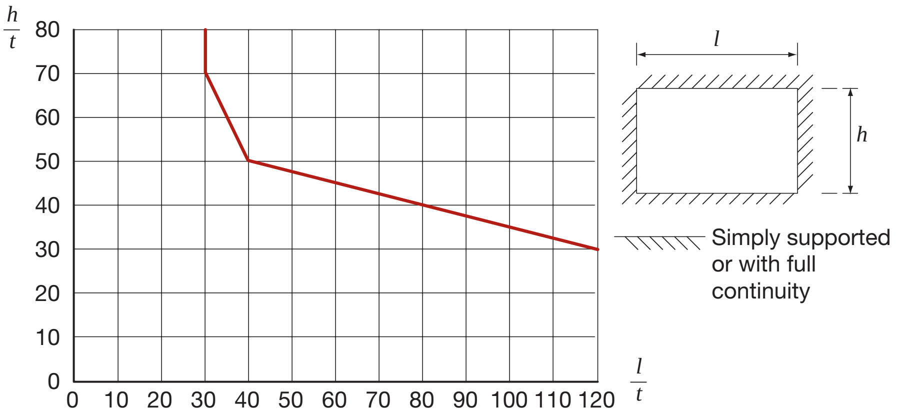


Fig 5.17 Limiting height and length to thickness ratios of walls restrained on all four edges

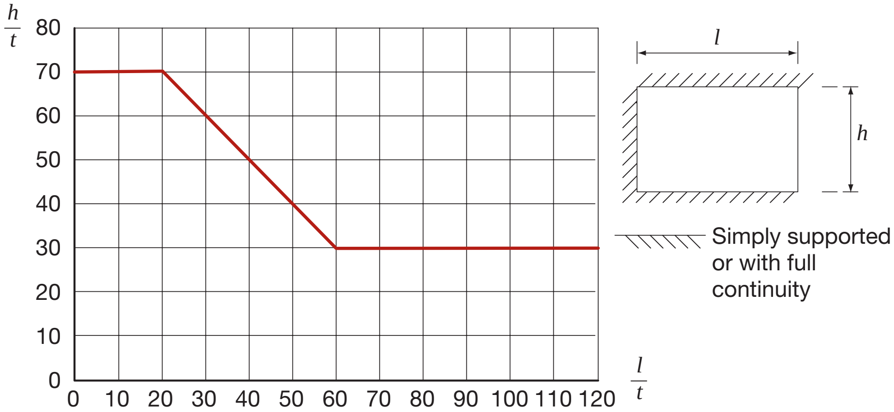


Fig 5.18 Limiting height and length to thickness ratios of walls restrained at the bottom, top and one vertical edge

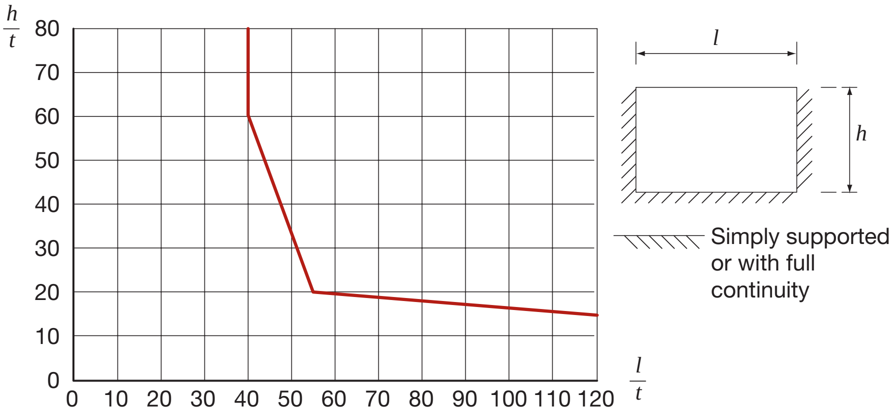


Fig 5.19 Limiting height and length to thickness ratios of walls restrained at the vertical edges and the bottom, but not the top

5.4.3 Direction of span and support conditions

Masonry is an anisotropic material. When used in its unreinforced form and subjected to bending, it has a greater flexural strength if the potential failure plane is perpendicular, rather than parallel, to the bed joints. As masonry units are commonly laid on their bed face then the two orthogonal flexural strengths referred to above relate to 'horizontally spanning' f_{xk2} and 'vertically spanning' f_{xk1} walls respectively. It is usually more economic to span masonry walls horizontally. Values of f_{xk2} and f_{xk1} may be obtained from Table 4.7.

The ultimate strength of the panel as a flexural member is governed by the capacity of the masonry to resist flexural tension. Any precompression present as a result of axial vertical loading will enhance the vertical spanning resistance of the wall to lateral loading. This enhancement of the vertical load carrying capacity will also influence the orthogonal ratio of the wall which is discussed in Section 5.4.4.

In some situations, where the masonry carries significant vertical loads, a check on compressive stresses will also be required.

The effects of any eccentricities in the vertical loads should also be considered, since they will induce additional moments in the masonry.

Vertical junctions between a panel under consideration and supporting return walls or columns may be fully bonded, tied or untied. Examples of such junctions and how they should be considered are shown in Figure 5.20. In a cavity wall, continuity across a vertical support may be assumed even if only one leaf is continuous over the support, provided that the cavity wall contains the recommended spacing of ties and that it is the thicker leaf that is continuous.

Horizontal supports at the top and bottom of a panel can be continuous or simply supported. If no support is available, the panel is considered to have a free edge. Some examples of different types of horizontal support are shown in Figure 5.20.

The effects of damp proof courses (dpc) need to be considered in laterally loaded masonry. Their presence complicates the design since they generally act as a discontinuity in a laterally loaded wall. Some continuity is, however, still possible because of gravity structural action, which will be considered later. Otherwise, a simply supported edge should be assumed. If the dpc has insufficient shear resistance, a free edge must be assumed.

The effects of movement joints need to be considered. Vertical movement joints may be tied or untied. An untied joint should be treated as a free edge. A tied joint may be sufficient to transfer shear but it is unlikely to be capable of transferring moment, and therefore at best should be treated as a simply supported edge.

Two vertical movement joints acting as free edges result in the panel spanning vertically, i.e. in its weaker direction. It is more efficient to use the horizontal spanning capability of masonry, and therefore it is preferable to position the vertical movement joints at vertical lateral support locations.

A horizontal movement joint at the top of a panel can be considered as either a free edge or a simply supported edge depending on the detail adopted, as full continuity is generally not easily achieved. Simple support can be achieved by provision of suitable floor restraint, by means of direct shear, ties or sliding anchors.

Consideration should be given to the ability of wall ties to transmit any out of balance tensile and compressive forces across the cavity. Declared strengths will be provided by manufacturers.

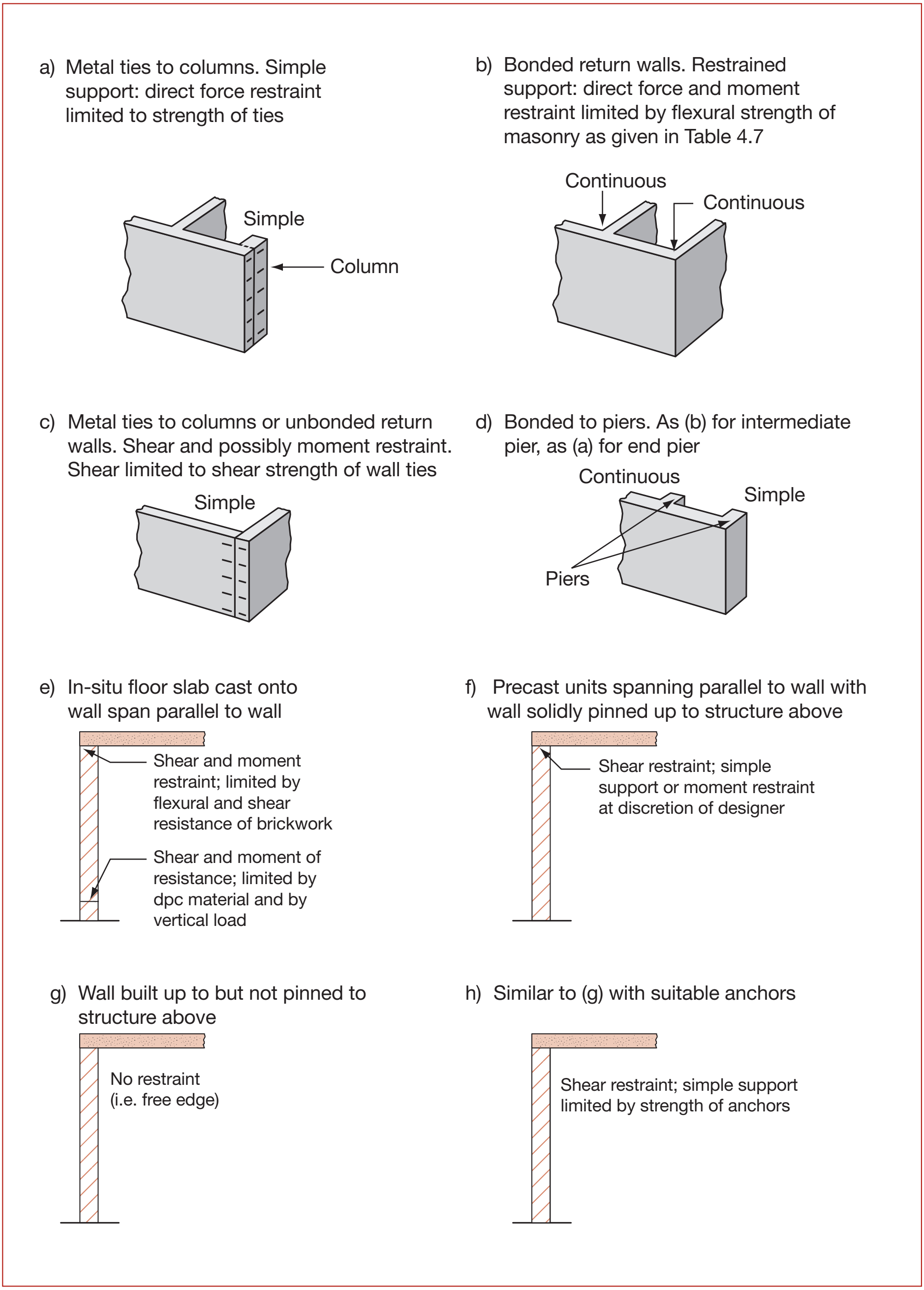


Fig 5.20 Continuous and simple edge conditions

5.4.4 Two-way spanning walls

Bending moments may be derived by yield-line theory. For panels without openings, the bending moments per unit length are:

$\alpha_2 W_{Ed} l^2$ when the plane of failure is perpendicular to the bed joints, and
$\mu\alpha_2 W_{Ed} l^2$ when the plane of failure is parallel to the bed joints.

Where: α_2 is the bending moment coefficient taken from Table 5.3
W_{Ed} is the design lateral load per unit area
l is the length of the panel between supports
μ is the orthogonal strength ratio, i.e. the ratio of flexural strength when failure is parallel to the bed joints to the flexural strength when failure is perpendicular to the bed joints (e.g. for concrete brick sized units in M4 mortar: μ= 0.3/0.9 = 1/3). Values for flexural strength are given in Table 4.7, as given in the UK National Annex[4].

Vertical loading increases the flexural strength of a panel in the parallel direction, in which case μ may be modified by using a flexural strength in the parallel direction of:

$$f_{xk1} + \gamma_M \sigma_d$$

Where: γ_M is the appropriate partial factor for materials taken from Table 4.9
σ_d is the design vertical dead load per unit area.

5.4.5 Openings

The guidance given above on the design of laterally loaded panels without openings is based on research, in which mainly storey height rectangular panels, without openings, were tested. When irregular shapes of panel, or those with substantial openings, are to be designed, it will often be possible to divide them into sub-panels (see Figure 5.21), which can then be calculated using the rules given above. Alternatively an analysis, using a recognised method of obtaining bending moments in flat plates, e.g. finite element or yield line, may be used, and these moments can then be used instead of the ones obtained from the coefficients given in Table 5.3. Figure 5.22 provides guidance on openings which can be accommodated without further calculation.

Small openings in panels will have little effect on the strength of the panel in which they occur, and they can be ignored. When suitable timber or metal frames are built into openings, the strength of the frame, taken in conjunction with the masonry panel, will often be sufficient to replace the strength lost by the area of the opening. This is largely a matter of engineering judgment.

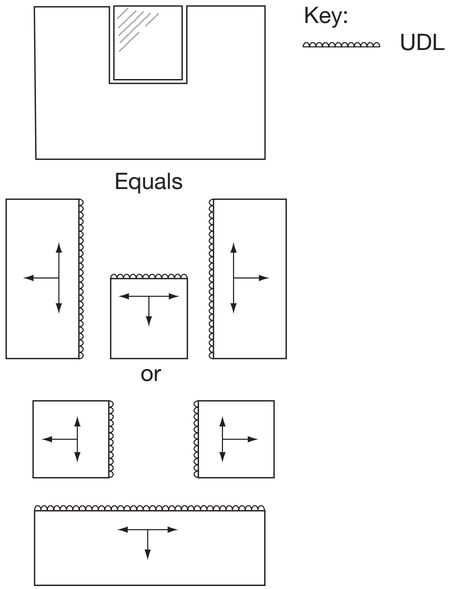
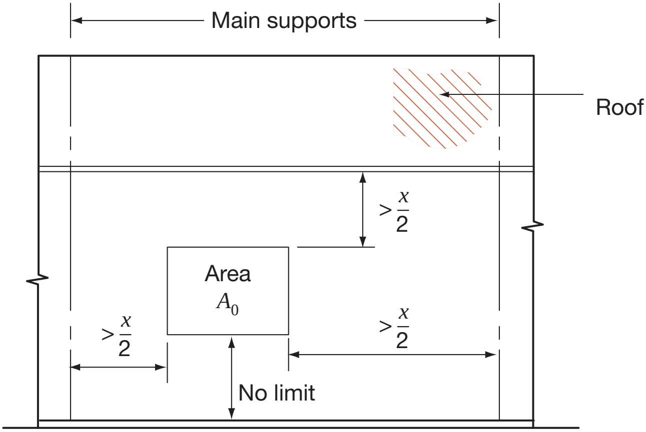


Fig 5.21 Example of subdivision of panel with openings



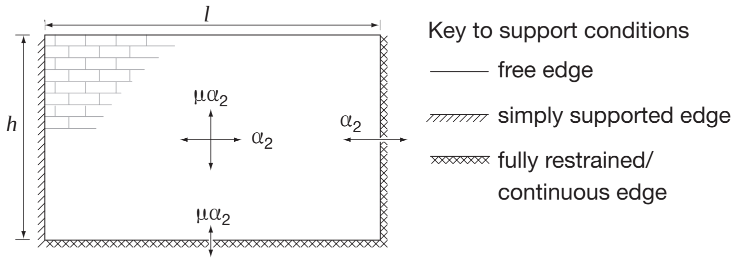
Notes

a x is the maximum dimension of opening (height or length).

b A_0 is the permitted area of opening which must not exceed 10% of the maximum allowable area of the panel or 25% of the actual area of the panel, whichever is the lesser.

Fig 5.22 Maximum size of opening in a wall without calculation

Table 5.3 Bending moment coefficients α_2 in single leaf laterally loaded wall panels of thickness less than or equal to 250mm

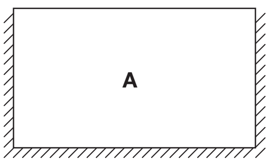


Notes

a α_2, $\mu\alpha_2$ are moment coefficients in the indicated directions.

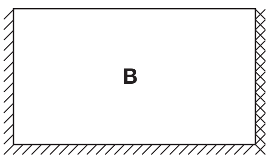
b This table is based on data from EC6 Part 1-1[1].

Wall support condition



μ	h/l 0.30	0.50	0.75	1.00	1.25	1.50	1.75	2.00
1.0	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	0.095
0.80	0.034	0.049	0.064	0.075	0.083	0.089	0.093	0.097
0.70	0.035	0.051	0.066	0.077	0.085	0.091	0.095	0.098
0.60	0.038	0.053	0.069	0.080	0.088	0.093	0.097	0.100
0.50	0.040	0.056	0.073	0.083	0.090	0.095	0.099	0.102
0.40	0.043	0.061	0.077	0.087	0.093	0.098	0.101	0.104
0.35	0.045	0.064	0.080	0.089	0.095	0.100	0.103	0.105
0.30	0.048	0.067	0.082	0.091	0.097	0.101	0.104	0.107
0.25	0.050	0.071	0.085	0.094	0.099	0.103	0.106	0.109
0.20	0.054	0.075	0.089	0.097	0.102	0.105	0.108	0.111
0.15	0.060	0.080	0.093	0.100	0.104	0.108	0.110	0.113
0.10	0.069	0.087	0.098	0.104	0.108	0.111	0.113	0.115
0.05	0.082	0.097	0.105	0.110	0.113	0.115	0.116	0.117

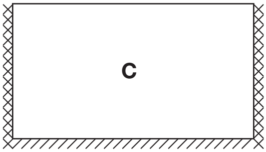
Wall support condition



μ	h/l 0.30	0.50	0.75	1.00	1.25	1.50	1.75	2.00
1.0	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	0.068
0.80	0.027	0.037	0.049	0.056	0.061	0.065	0.067	0.069
0.70	0.028	0.039	0.051	0.058	0.062	0.066	0.068	0.070
0.60	0.030	0.042	0.053	0.059	0.064	0.067	0.069	0.071
0.50	0.031	0.044	0.055	0.061	0.066	0.069	0.071	0.072
0.40	0.034	0.047	0.057	0.063	0.067	0.070	0.072	0.074
0.35	0.035	0.049	0.059	0.065	0.068	0.071	0.073	0.074
0.30	0.037	0.051	0.061	0.066	0.070	0.072	0.074	0.075
0.25	0.039	0.053	0.062	0.068	0.071	0.073	0.075	0.077
0.20	0.043	0.056	0.065	0.069	0.072	0.074	0.076	0.078
0.15	0.047	0.059	0.067	0.071	0.074	0.076	0.077	0.079
0.10	0.052	0.063	0.070	0.074	0.076	0.078	0.079	0.080
0.05	0.060	0.069	0.074	0.077	0.079	0.080	0.081	0.082

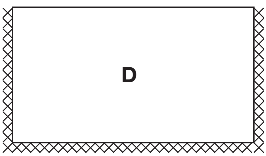
Table 5.3 (Continued)

Wall support condition



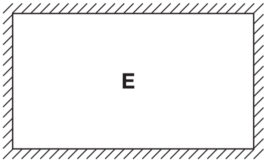
μ	h/l 0.30	0.50	0.75	1.00	1.25	1.50	1.75	2.00
1.0	0.020	0.028	0.037	0.042	0.045	0.048	0.050	0.051
0.90	0.021	0.029	0.038	0.043	0.046	0.048	0.050	0.052
0.80	0.022	0.031	0.039	0.043	0.047	0.049	0.051	0.052
0.70	0.023	0.032	0.040	0.044	0.048	0.050	0.051	0.053
0.60	0.024	0.034	0.041	0.046	0.049	0.051	0.052	0.053
0.50	0.025	0.035	0.043	0.047	0.050	0.052	0.053	0.054
0.40	0.027	0.038	0.044	0.048	0.051	0.053	0.054	0.055
0.35	0.029	0.039	0.045	0.049	0.052	0.053	0.054	0.055
0.30	0.030	0.040	0.046	0.050	0.052	0.054	0.055	0.056
0.25	0.032	0.042	0.048	0.051	0.053	0.054	0.056	0.057
0.20	0.034	0.043	0.049	0.052	0.054	0.055	0.056	0.058
0.15	0.037	0.046	0.051	0.053	0.055	0.056	0.057	0.059
0.10	0.041	0.048	0.053	0.055	0.056	0.057	0.058	0.059
0.05	0.046	0.052	0.055	0.057	0.058	0.059	0.059	0.060

Wall support condition



μ	h/l 0.30	0.50	0.75	1.00	1.25	1.50	1.75	2.00
1.0	0.013	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	0.048
0.80	0.015	0.023	0.032	0.038	0.041	0.044	0.047	0.048
0.70	0.016	0.025	0.033	0.039	0.043	0.045	0.047	0.049
0.60	0.017	0.026	0.035	0.040	0.044	0.046	0.048	0.050
0.50	0.018	0.028	0.037	0.042	0.045	0.048	0.050	0.051
0.40	0.020	0.031	0.039	0.043	0.047	0.049	0.051	0.052
0.35	0.022	0.032	0.040	0.044	0.048	0.050	0.051	0.053
0.30	0.023	0.034	0.041	0.046	0.049	0.051	0.052	0.053
0.25	0.025	0.035	0.043	0.047	0.050	0.052	0.053	0.054
0.20	0.027	0.038	0.044	0.048	0.051	0.053	0.054	0.055
0.15	0.030	0.040	0.046	0.050	0.052	0.054	0.055	0.056
0.10	0.034	0.043	0.049	0.052	0.054	0.055	0.056	0.057
0.05	0.041	0.048	0.053	0.055	0.056	0.057	0.058	0.059

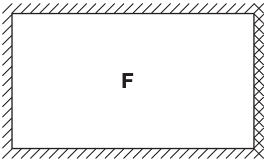
Wall support condition



μ	h/l 0.30	0.50	0.75	1.00	1.25	1.50	1.75	2.00
1.0	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	0.074
0.80	0.010	0.021	0.035	0.046	0.056	0.064	0.071	0.076
0.70	0.011	0.023	0.037	0.049	0.059	0.067	0.073	0.078
0.60	0.012	0.025	0.040	0.053	0.062	0.070	0.076	0.081
0.50	0.014	0.028	0.044	0.057	0.066	0.074	0.080	0.085
0.40	0.017	0.032	0.049	0.062	0.071	0.078	0.084	0.088
0.35	0.018	0.035	0.052	0.064	0.074	0.081	0.086	0.090
0.30	0.020	0.038	0.055	0.068	0.077	0.083	0.089	0.093
0.25	0.023	0.042	0.059	0.071	0.080	0.087	0.091	0.096
0.20	0.026	0.046	0.064	0.076	0.084	0.090	0.095	0.099
0.15	0.032	0.053	0.070	0.081	0.089	0.094	0.098	0.103
0.10	0.039	0.062	0.078	0.088	0.095	0.100	0.103	0.106
0.05	0.054	0.076	0.090	0.098	0.103	0.107	0.109	0.110

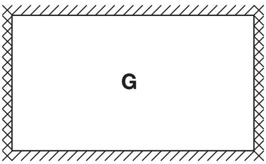
Table 5.3 (Continued)

Wall support condition



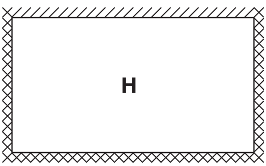
μ	h/l							
	0.30	**0.50**	**0.75**	**1.00**	**1.25**	**1.50**	**1.75**	**2.00**
1.0	0.008	0.016	0.026	0.034	0.041	0.046	0.051	0.054
0.90	0.008	0.017	0.027	0.036	0.042	0.048	0.052	0.055
0.80	0.009	0.018	0.029	0.037	0.044	0.049	0.054	0.057
0.70	0.010	0.020	0.031	0.039	0.046	0.051	0.055	0.058
0.60	0.011	0.022	0.033	0.042	0.048	0.053	0.057	0.060
0.50	0.013	0.024	0.036	0.044	0.051	0.056	0.059	0.062
0.40	0.015	0.027	0.039	0.048	0.054	0.058	0.062	0.064
0.35	0.016	0.029	0.041	0.050	0.055	0.060	0.063	0.066
0.30	0.018	0.031	0.044	0.052	0.057	0.062	0.065	0.067
0.25	0.020	0.034	0.046	0.054	0.060	0.063	0.066	0.069
0.20	0.023	0.037	0.049	0.057	0.062	0.066	0.068	0.070
0.15	0.027	0.042	0.053	0.060	0.065	0.068	0.070	0.072
0.10	0.032	0.048	0.058	0.064	0.068	0.071	0.073	0.074
0.05	0.043	0.057	0.066	0.070	0.073	0.075	0.077	0.078

Wall support condition



μ	h/l							
	0.30	**0.50**	**0.75**	**1.00**	**1.25**	**1.50**	**1.75**	**2.00**
1.0	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	0.043
0.80	0.008	0.016	0.024	0.031	0.035	0.039	0.042	0.044
0.70	0.009	0.017	0.026	0.032	0.037	0.040	0.043	0.045
0.60	0.010	0.019	0.028	0.034	0.038	0.042	0.044	0.046
0.50	0.011	0.021	0.030	0.036	0.040	0.043	0.046	0.048
0.40	0.013	0.023	0.032	0.038	0.042	0.045	0.047	0.049
0.35	0.014	0.025	0.033	0.039	0.043	0.046	0.048	0.050
0.30	0.016	0.026	0.035	0.041	0.044	0.047	0.049	0.051
0.25	0.018	0.028	0.037	0.042	0.046	0.048	0.050	0.052
0.20	0.020	0.031	0.039	0.044	0.047	0.050	0.052	0.054
0.15	0.023	0.034	0.042	0.046	0.049	0.051	0.053	0.055
0.10	0.027	0.038	0.045	0.049	0.052	0.053	0.055	0.057
0.05	0.035	0.044	0.050	0.053	0.055	0.056	0.057	0.058

Wall support condition



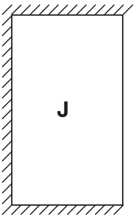
μ	h/l							
	0.30	**0.50**	**0.75**	**1.00**	**1.25**	**1.50**	**1.75**	**2.00**
1.0	0.005	0.011	0.018	0.024	0.029	0.033	0.036	0.039
0.90	0.006	0.012	0.019	0.025	0.030	0.034	0.037	0.040
0.80	0.006	0.013	0.020	0.027	0.032	0.035	0.038	0.041
0.70	0.007	0.014	0.022	0.028	0.033	0.037	0.040	0.042
0.60	0.008	0.015	0.024	0.030	0.035	0.038	0.041	0.043
0.50	0.009	0.017	0.025	0.032	0.036	0.040	0.043	0.045
0.40	0.010	0.019	0.028	0.034	0.039	0.042	0.045	0.047
0.35	0.011	0.021	0.029	0.036	0.040	0.043	0.046	0.047
0.30	0.013	0.022	0.031	0.037	0.041	0.044	0.047	0.049
0.25	0.014	0.024	0.033	0.039	0.043	0.046	0.048	0.051
0.20	0.016	0.027	0.035	0.041	0.045	0.047	0.049	0.052
0.15	0.019	0.030	0.038	0.043	0.047	0.049	0.051	0.053
0.10	0.023	0.034	0.042	0.047	0.050	0.052	0.053	0.054
0.05	0.031	0.041	0.047	0.051	0.053	0.055	0.056	0.056

Table 5.3 (Continued)

Wall support condition

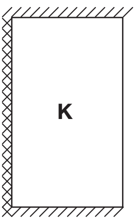
μ	h/l 0.30	0.50	0.75	1.00	1.25	1.50	1.75	2.00
1.0	0.004	0.009	0.015	0.021	0.026	0.030	0.033	0.036
0.90	0.004	0.010	0.016	0.022	0.027	0.031	0.034	0.037
0.80	0.005	0.010	0.017	0.023	0.028	0.032	0.035	0.038
0.70	0.005	0.011	0.019	0.025	0.030	0.033	0.037	0.039
0.60	0.006	0.013	0.020	0.026	0.031	0.035	0.038	0.041
0.50	0.007	0.014	0.022	0.028	0.033	0.037	0.040	0.042
0.40	0.008	0.016	0.024	0.031	0.035	0.039	0.042	0.044
0.35	0.009	0.017	0.026	0.032	0.037	0.040	0.043	0.045
0.30	0.010	0.019	0.028	0.034	0.038	0.042	0.044	0.046
0.25	0.011	0.021	0.030	0.036	0.040	0.043	0.046	0.048
0.20	0.013	0.023	0.032	0.038	0.042	0.045	0.047	0.050
0.15	0.016	0.026	0.035	0.041	0.044	0.047	0.049	0.051
0.10	0.020	0.031	0.039	0.044	0.047	0.050	0.052	0.054
0.05	0.027	0.038	0.045	0.049	0.052	0.053	0.055	0.056

Wall support condition



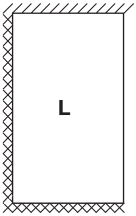
μ	h/l 0.30	0.50	0.75	1.00	1.25	1.50	1.75	2.00
1.0	0.009	0.023	0.046	0.071	0.096	0.122	0.151	0.180
0.90	0.010	0.026	0.050	0.076	0.103	0.131	0.162	0.193
0.80	0.012	0.028	0.054	0.083	0.111	0.142	0.175	0.208
0.70	0.013	0.032	0.060	0.091	0.121	0.156	0.191	0.227
0.60	0.015	0.036	0.067	0.100	0.135	0.173	0.211	0.250
0.50	0.018	0.042	0.077	0.113	0.153	0.195	0.237	0.280
0.40	0.021	0.050	0.090	0.131	0.177	0.225	0.272	0.321
0.35	0.024	0.055	0.098	0.144	0.194	0.244	0.296	0.347
0.30	0.027	0.062	0.108	0.160	0.214	0.269	0.325	0.381
0.25	0.032	0.071	0.122	0.180	0.240	0.300	0.362	0.428
0.20	0.038	0.083	0.142	0.208	0.276	0.344	0.413	0.488
0.15	0.048	0.100	0.173	0.250	0.329	0.408	0.488	0.570
0.10	0.065	0.131	0.224	0.321	0.418	0.515	0.613	0.698
0.05	0.106	0.208	0.344	0.482	0.620	0.759	0.898	0.959

Wall support condition



μ	h/l 0.30	0.50	0.75	1.00	1.25	1.50	1.75	2.00
1.0	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	0.129
0.80	0.011	0.025	0.045	0.065	0.084	0.103	0.120	0.136
0.70	0.012	0.028	0.049	0.070	0.091	0.110	0.128	0.145
0.60	0.014	0.031	0.054	0.077	0.099	0.119	0.138	0.155
0.50	0.016	0.035	0.061	0.085	0.109	0.130	0.149	0.167
0.40	0.019	0.041	0.069	0.097	0.121	0.144	0.164	0.182
0.35	0.021	0.045	0.075	0.104	0.129	0.152	0.173	0.191
0.30	0.024	0.050	0.082	0.112	0.139	0.162	0.183	0.202
0.25	0.028	0.056	0.091	0.123	0.150	0.174	0.196	0.217
0.20	0.033	0.064	0.103	0.136	0.165	0.190	0.211	0.234
0.15	0.040	0.077	0.119	0.155	0.184	0.210	0.231	0.253
0.10	0.053	0.096	0.144	0.182	0.213	0.238	0.260	0.279
0.05	0.080	0.136	0.190	0.230	0.260	0.286	0.306	0.317

Table 5.3 (Continued)

Wall support condition	μ	h/l							
		0.30	**0.50**	**0.75**	**1.00**	**1.25**	**1.50**	**1.75**	**2.00**
	**1.0**	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	0.107
	0.80	0.008	0.018	0.034	0.051	0.067	0.084	0.099	0.114
	0.70	0.009	0.021	0.038	0.056	0.073	0.090	0.106	0.122
	0.60	0.010	0.023	0.042	0.061	0.080	0.098	0.115	0.131
	0.50	0.012	0.027	0.048	0.068	0.089	0.108	0.126	0.142
	0.40	0.014	0.032	0.055	0.078	0.100	0.121	0.139	0.157
	0.35	0.016	0.035	0.060	0.084	0.108	0.129	0.148	0.165
	0.30	0.018	0.039	0.066	0.092	0.116	0.138	0.158	0.176
	0.25	0.021	0.044	0.073	0.101	0.127	0.150	0.170	0.190
	0.20	0.025	0.052	0.084	0.114	0.141	0.165	0.185	0.206
	0.15	0.031	0.061	0.098	0.131	0.159	0.184	0.205	0.226
	0.10	0.041	0.078	0.121	0.156	0.186	0.212	0.233	0.252
	0.05	0.064	0.114	0.164	0.204	0.235	0.260	0.281	0.292

5.4.6 One-way spanning walls

Where walls span purely vertically or horizontally, or predominantly vertically or horizontally, they may be designed in accordance with Section 5.4.4 by ignoring two-way action and using simple single span bending moments. Vertically spanning walls may be treated as propped cantilevers, as described below. Where suitable in-plane abutment forces can be generated along vertical or horizontal panel edges, the wall panel can be treated as a shallow arch to resist lateral loading, see Section 5.4.9.

Walls spanning vertically may be treated as simply supported at the top and generally partially fixed at the base. The partial base fixity is generated by the gravity action of the self-weight of the wall and any permanently imposed dead loads on it. The wall is therefore considered as a 'cracked section' and in these circumstances no additional tensile strength should be taken into account at this level. Where clay damp proof course units to BS EN 771-1[18] are used the flexural strength of the masonry may be allowed for at the base of the wall.

The resistance moment at the base, generated by gravity action, is termed the stability moment of resistance, M_{Rds}. If M_{Rds} exceeds $W_{Ed}\,h^2/8$, that of a propped cantilever, the wall should be designed as a propped cantilever (see Figure 5.23).

The moments of resistance at the base and within the wall height are calculated as follows.

Where: W_{Ed} is the design lateral load per unit area
h is the height of the panel.

but $h \leqslant 30t_{ef}$ for simple support condition

Where: t_{ef} is the effective thickness of the wall panel.

The stability moment of resistance at base of wall is shown in Figure 5.24.

Where: N_{id} is the design dead load ($= G_k \gamma_{G,inf}$)
f_k is the characteristic compressive strength of masonry
γ_M is the appropriate partial factor for materials
z is the lever arm
b_{ci} is the width of stressed area
G_k is the characteristic value of the permanent action (dead load)
$\gamma_{G,inf}$ is the partial factor for G_k (adverse).

Stability moment of resistance at base is given by:

$$M_{Rds} = N_{id}\ z$$

The minimum width of wall stressed to its ultimate strength b_{ci} creates the maximum lever arm, about which the dead load of the wall rotates to generate the maximum stability moment of resistance.

Within the height of the wall, the design moment of resistance is given by:

$$M_{Rd} = \left(\frac{f_{xk1}}{\gamma_M} + \sigma_d\right)Z$$

Where: f_{xk1} is the characteristic flexural strength of masonry bending about an axis parallel to bed joints
γ_M is the appropriate partial factor for materials
σ_d is the design vertical dead load per unit area
Z is the section modulus of the plan shape of the wall; which may take into account any variations in the wall.

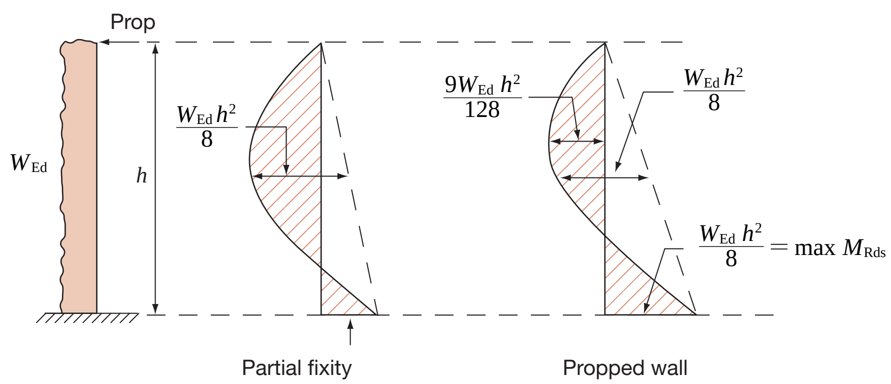
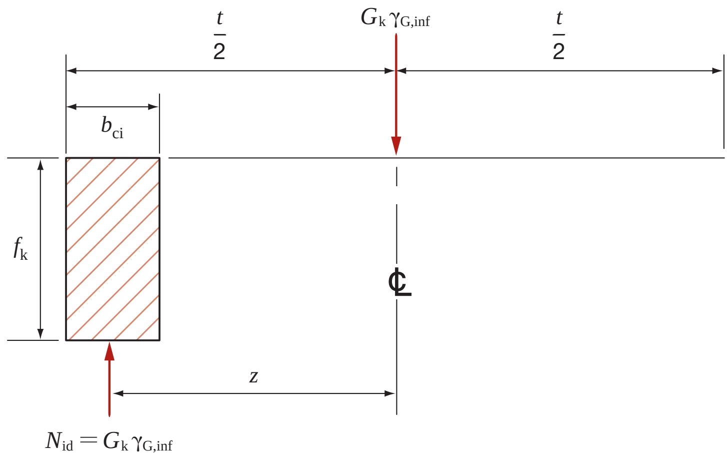


Fig 5.23 Moment diagrams for vertically spanning walls



Note

$\gamma_{G,inf}$ is taken as favourable, i.e. = 1.0.

Fig 5.24 Stability moment of resistance at base of wall

In calculating N_{id} and σ_d account should be taken of any uplift forces due to the wind loading on the roof of the building.

In practice, where fixity is developed by dead loads other than the self-weight of the wall, the wind load is generally unlikely to be critical. This situation might occur in a multi-storey structure where fixity may be developed at the top of the storey height because of the stability moment of resistance across the 'cracked section' that results from the loads supported from above. The condition described in Figure 5.23 relates to a single-storey structure or the top storey in a multi-storey building.

The design analysis relating to Figure 5.23 is considered to provide a practical solution. An additional 'cracked section' check for stability in the height (upper level) of the wall is required. This additional check is based upon the variation of flexural strengths between clay and concrete masonry.

Under the ultimate limit state the member must satisfy the following conditions.

- The design flexural tensile stress (other than in the vicinity of the lower support) should not exceed the appropriate value of design flexural strength. At the lower support, the design moment of resistance should be assessed using the appropriate partial factors.
- The wall should be stable when its strength is derived from gravity forces only (i.e. when its flexural tensile strength is ignored).
- The design stress in the compressive zone of the wall should not exceed the design compressive strength.

Walls spanning horizontally are treated in a similar manner; in the formulae and limitations outlined earlier in this subsection substitute l for h (where l is the length of the panel), and use f_{xk2} (bending perpendicular to the bed joints); in practice, σ_d would be taken as zero. There is no base stability moment to take advantage of, although if the wall is continuous across its vertical supports it may be possible to design it as a continuous beam. It is important to check that the location of vertical movement joints (particularly in concrete blockwork walls) has been considered in the analysis of continuous beam bending moments.

5.4.7 Cavity walls

Provided that the wall ties used are capable of transmitting the forces to which they are subjected, the design lateral strength of a cavity wall may be taken as the sum of the design strengths of the two individual leaves, allowing for the additional strength of any piers bonded to one or both of the leaves.

5.4.8 Geometric walls

EC6 Part 1-1[1] does not specifically deal with the design of geometric walls subject to lateral loading, such as diaphragm and fin walls (see Figures 5.25 and 5.26). However, UK practice[60] for the design procedure for these walls is essentially the same at that given in Section 5.4.5 for vertically spanning walls.

The geometric profile of the wall provides considerable enhancement to its resistance to lateral load at base level, with an increased lever arm for the gravitational mass, and within the wall height, with an increased section modulus to minimise tensile stresses. Note that this *Manual* assumes that the building does not rely on the flexural strength of these walls for its overall stability.

In assessing the section modulus of a geometric wall, the outstanding length of flange from the face of the fin or cross-rib should be taken as four times the thickness of the flange when the flange is unrestrained, or six times the thickness of the flange when the flange is continuous, but in no case more than half the clear distance between fins or cross-ribs.

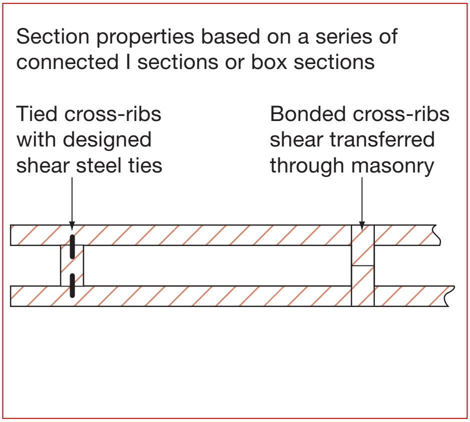


Fig 5.25 Typical diaphragm wall section

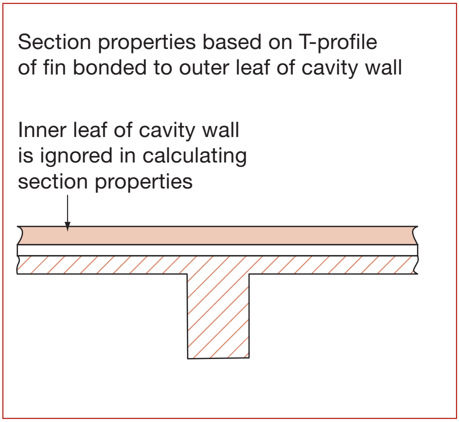


Fig 5.26 Typical fin wall section

A suggested design procedure applicable to both diaphragm and fin walls is given below.

i) Calculate loadings (dead, imposed and wind).

ii) Select trial section of wall profile and masonry strength; suggested trial section selection procedures for both diaphragm and fin walls are given in Reference 60.

iii) Calculate applied bending moment at base of wall and compare with stability moment of resistance M_{Rds}.

iv) Calculate position and magnitude of maximum applied bending moment within height of wall and compare with flexural resistance of wall at this level.

v) Check that the wall is stable when its strength is derived from gravity forces alone (i.e. ignoring the flexural tensile strength of the masonry) with partial factors of 1.0 on the permanent and wind actions.

vi) Calculate shear stresses at junctions of cross-ribs (in diaphragm walls) and fin/flange (in fin walls).

vii) Design shear ties or calculate shear resistance of the bonded masonry at these shear interfaces.

Note that because of the nature of differential movements between clay units and concrete units the mixing of these units within geometric wall profiles should be considered with caution.

Note also that the deflection of the structure providing horizontal support to the top of the wall should be taken into account in calculating the bending moments in the wall.

The design procedure is one of trial and error.

5.4.9 Arching

The development of lateral load resistance through the in-plane arching action of a wall panel may be considered. Shrinkage effects in concrete blockwork and potentially inadequate frame support details, etc. can make this approach unreliable, and it is not, in general, recommended.

However, where a masonry wall is built solidly between supports (vertical or horizontal) which are capable of resisting an arch thrust without significant deflection, movement or distortion, the wall may be designed to resist lateral loading applied to it by assuming that it behaves as a three-pin arch. Figure 5.27 illustrates the structural model to be adopted.

Thus, the arch rise, r, is given by:

$$r = 0.9t - d_{\mathrm{a}}$$

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.

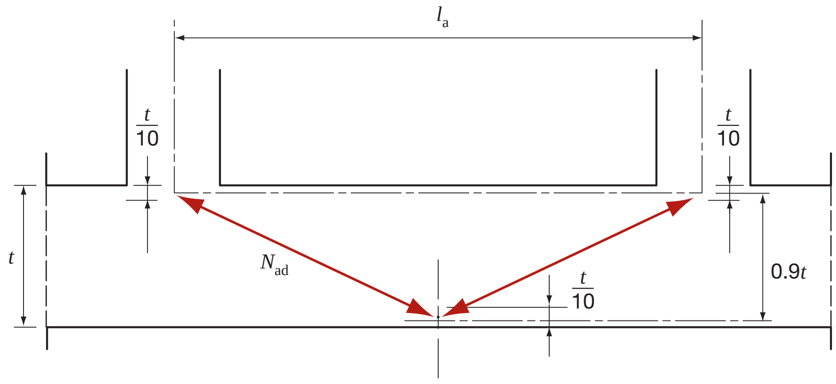


Fig 5.27 Arch assumed for resisting lateral loads (diagrammatic)

The maximum design arch thrust per unit length of wall N_{ad} may be obtained from:

$$N_{ad} = 1.5 f_d \frac{t}{10}$$

and where the lateral deflection is small, the design lateral strength is given by:

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

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

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 0.1N/mm^2
- the slenderness ratio does not exceed 20.

5.4.10 Bed joint reinforcement

This *Manual*, whilst dealing with the design of plain masonry, also considers the use of bed joint reinforcement to enhance the crack resistance of long lengths of masonry and to enhance the lateral load resistance of masonry walls. Neither of these aspects is covered by Part 1-1 of EC6[1] specifically, although the design procedures for reinforced masonry members can be used to reinforce masonry to resist lateral loading. However, the very low percentages of reinforcement involved with bed joint reinforcement and the similarly low lever arms involved make such a design of little practical use.

UK practice is to follow empirical rules which allow the unreinforced lateral resistance of plain masonry walls to be enhanced by up to 50%. The most effective method of design of these walls follows the method for unreinforced panel design, as given in Section 5.4.4, but uses a modified orthogonal ratio.

For leaves which contain bed joint reinforcement, the orthogonal ratio is defined as the ratio of the moment of resistance about a horizontal axis, that is when the plane of failure is parallel to a bed joint, to the moment of resistance about a vertical axis, that is when the plane of failure is perpendicular to a bed joint. The moment of resistance about the horizontal axis is given by:

$$\frac{f_{xk1}Z}{\gamma_M}$$

Where: f_{xk1} is the characteristic flexural strength of the masonry when the plane of failure is parallel to the bed joints given in Table 4.7

γ_M is the partial factor for strength of masonry given in Table 4.9

Z is the section modulus per unit length of the bed joint.

The design moment of resistance about the vertical axis is as given in EC6 Part 1-1[1] Clause 6.6.2(4). The design moment in the panel is found using the appropriate bending moment coefficient in Table 5.3. Equating the design moment with the design moment of resistance enables the lateral load capacity of the wall to be obtained.

For cavity walls the recommendations of EC6 Part 1-1[1] Clause 6.3.1(6) should be followed.

The maximum enhancement of lateral load resistance above that for the equivalent unreinforced walls should be taken to be 50%, unless a serviceability and deflection check is carried out.

5.5 Serviceability limit state

It is a principle of EC6 Part 1-1[1] that the serviceability limit state shall be considered for all aspects of the structure including ancillary components in the masonry (see Section 2.2(2) in EC6). In general, no separate check is required in unreinforced masonry for the serviceability limit states of cracking and deflection; satisfying the ultimate limit state usually being sufficient to achieve adequate performance under service conditions. Certain clauses in EC6 contain simplified rules in relation to serviceability limit states, e.g. limiting panel sizes for laterally loaded masonry and limiting span depth ratios for reinforced masonry elements. Attention is drawn in EC6 to the need to consider in design the behaviour of other structural elements, such as deformations of floors, which may have an adverse effect upon masonry elements. Limits on the maximum spacing of movement joints also seek to control cracking in masonry walls. Where slenderness ratios exceed 27 (laterally loaded walls only), particular attention should be paid to the serviceability limit states.

6.1 Sequence of building and how it affects design

As with all structures, constructional details should be considered at the design stage. The majority of loadbearing masonry buildings rely, for their overall stability, on structural elements (e.g. roofs and floors) that will be installed after completion of the masonry elements. The design should therefore avoid (if possible) excessively slender or unbuttressed masonry members that, although stable in the final building, will require temporary propping during construction. If such members cannot be avoided, their need for temporary support should be made clear (in the Health and Safety plan and file) to all involved in the project, particularly the contractor and to the end user who may wish to alter or demolish the building.

6.2 Masonry details

6.2.1 Minimum dimensions

EC6 Part 1-1[1] gives limited guidance on minimum dimensions for masonry elements. The guidance offered for the minimum thickness of a wall is that it shall be sufficient to give a robust wall to satisfy the calculated requirement for thickness. As referred to in Section 1.3 the minimum net plan area for a loadbearing wall is $0.04m^2$ (after allowing for chases and recesses). It should be borne in mind that as far as EC6 is concerned a masonry pier is simply a short wall.

It would seem reasonable to use t_{ef} (effective thickness) in calculating the area of a cavity wall, otherwise a 1½ brick unit long cavity pier cannot be designed.

6.2.2 Bonding of masonry

The integrity and strength of masonry construction relies, in large measure, on the interlocking or overlapping of its units during construction; this is known as bonding of the masonry. Single unit thick walls are usually built in stretcher bond, and thicker walls in a range of bonds that include various arrangements of headers and stretchers (e.g. English or Flemish bonds). The importance of maintaining the bond and of filling the joint between the units with mortar has been discussed in Section 3.4.2.

Solid walls may also be built in the form of two single leaf walls, with a continuous joint between them, with ties or bed joint reinforcement across the central joint, such that the two leaves behave compositely. EC6 calls these double-leaf walls. They are known as collar jointed walls in the UK. The joint between the leaves should not be thicker than 25mm and UK practice is that the joint should be fully filled with mortar as the work proceeds (EC6 allows the joint to be unfilled). The UK National Annex[4] to EC6 Part 1-1 gives the minimum thickness of a solid single leaf loadbearing wall as **90mm**.

Cavity walls may have leaves of similar or different sized units. The thickness of each leaf should not be less than **75mm** (NA to EC6 Part 1-1[4]), with a cavity not normally less than 50mm in width. The leaves should be adequately tied together, see Section 6.3.

Stack bonding and similar forms of masonry construction, which introduce continuous vertical joints into the masonry, require bed joint reinforcement to ensure the integrity of the masonry. Bed joint reinforcement can also be used for crack control and to enhance the lateral resistance of masonry, see Section 5.4.10.

6.2.3 Mortar joints

EC6 Part 1-1[1] refers to general purpose masonry mortars, thin layer mortars and lightweight masonry mortars. General purpose and lightweight mortars are principally used in joints which are nominally 10mm thick. Thin joint mortars are used in bed joints with a thickness of between 0.5mm and 3mm. It is thus important that the masonry units with which these mortars are used are dimensionally co-ordinated in order to minimise the cutting of the masonry units. In cavity walls in particular, coursing levels in the two leaves should be co-ordinated to avoid problems with the fixing of wall ties. If this is not possible frame anchor type ties, plugged and screwed to the inner leaf, or other suitable ties can be used.

In masonry with normal thickness bed joints, over thick joints should be avoided, as they can reduce the loadbearing capacity of the masonry and the weathertightness of external masonry.

Soldier courses are commonly used as an architectural feature over windows or to define storey heights. They are likely to have less resistance to vertical loading than horizontally laid units, because of the change of aspect ratio with respect to the direction of loading. Similarly they may have a reduced resistance to horizontal loads because of their lack of proper (overlapping) bond.

When soldier courses are required, it may be necessary to use a slightly stiffer mortar to stop the masonry overturning on the bed joint and so that the mortar applied to the vertical unit face does not fall away before the unit is placed.

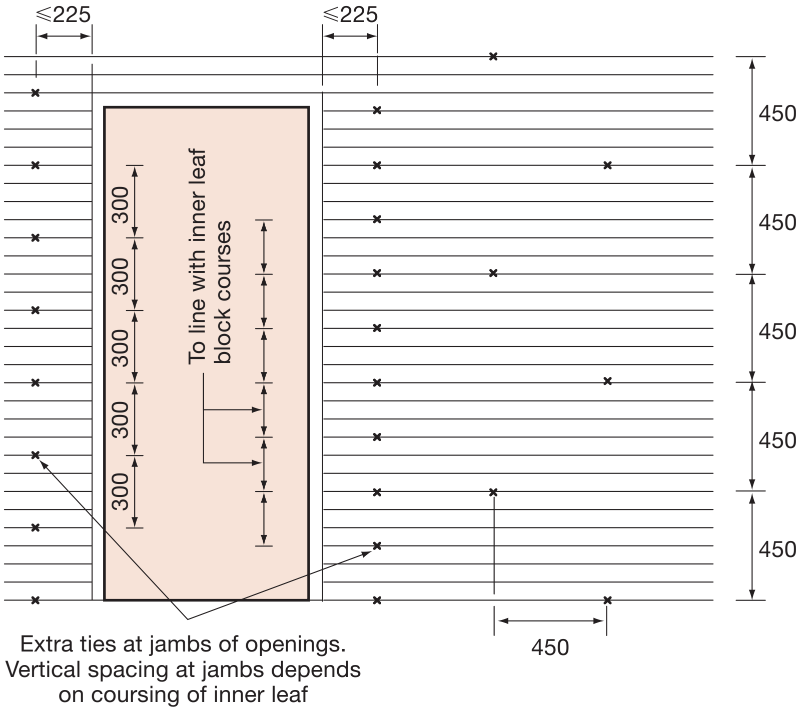
6.3 Connection of walls

Where walls are connected together with wall ties, the ties must be of the appropriate structural performance and durability for the particular circumstance. Figure 6.1 illustrates UK practice for the spacing of cavity wall ties. See also Section 5.3.4.

6.4 Chases and recesses

Chases and recesses are commonly formed in masonry walls to accommodate services etc. They can have a significant effect upon the stability of the wall in which they are formed. They can also affect the loadbearing capacity of the masonry and should therefore be allowed for in design where appropriate. EC6 Part 1-1[1] does not permit chases or recesses which are greater than half the shell thickness of the unit unless verified by calculation. Chases and services must not pass through structural elements such as lintels unless allowed for in their design. In cavity walls each leaf should be considered separately with respect to chases and recesses.

Table 6.1 gives maximum sizes of vertical chases and recesses which are permitted in masonry without further calculation by the UK National Annex to EC6 Part 1-1[4].



Note
All dimensions are in millimetres.

Fig 6.1 Spacing of cavity wall ties

In general horizontal and inclined chases should be avoided in masonry walls. Where they cannot be avoided, EC6 requires them to be located within one eighth of the clear height of the wall above or below floor level. Table 6.2 gives maximum sizes for horizontal and inclined chases which are permitted in masonry without further calculation, again by the UK National Annex to EC6 Part 1-1[4].

Where the limits in Tables 6.1 and 6.2 are exceeded, the vertical load, shear and flexural resistances of the masonry should be checked by calculation using the reduced section.

The requirements of Tables 6.1 and 6.2 are illustrated in Figures 6.2 and 6.3. For limitations on chases in non-loadbearing masonry walls which are fire resisting, see Section 7.7.

Table 6.1 Sizes of vertical chases and recesses in masonry, allowed without calculation

	Chases and recesses formed after construction of masonry		Chases and recesses formed during construction of masonry	
Thickness of wall t (mm)	max depth $t_{ch,v}$ (mm)	max width w_c (mm)	minimum wall thickness remaining t_r (mm)	max width w_c (mm)
75 – 89	30	75	60	300
90 – 115	30	100	70	300
116 – 175	30	125	90	300
176 – 225	30	150	140	300
226 – 300	30	175	175	300
> 300	30	200	215	300

Notes

a The maximum depth of the recess or chase should include the depth of any hole reached when forming the recess or chase.

b Vertical chases that do not extend more than one third of the storey height above floor level may have a depth of up to 80mm and a width of up to 120mm, if the thickness of the wall is 225mm or more.

c The horizontal distance between adjacent chases or between a chase and recess or an opening should not be less than 225mm.

d The horizontal distance between any two adjacent recesses, whether they occur on the same side or on opposite sides of the wall, or between a recess and an opening, should not be less than twice the width of the wider of the two recesses.

e The cumulative width of vertical chases and recesses should not exceed 0.13 times the length of the wall.

f This table is based on data from NA[4] to EC6 Part 1-1.

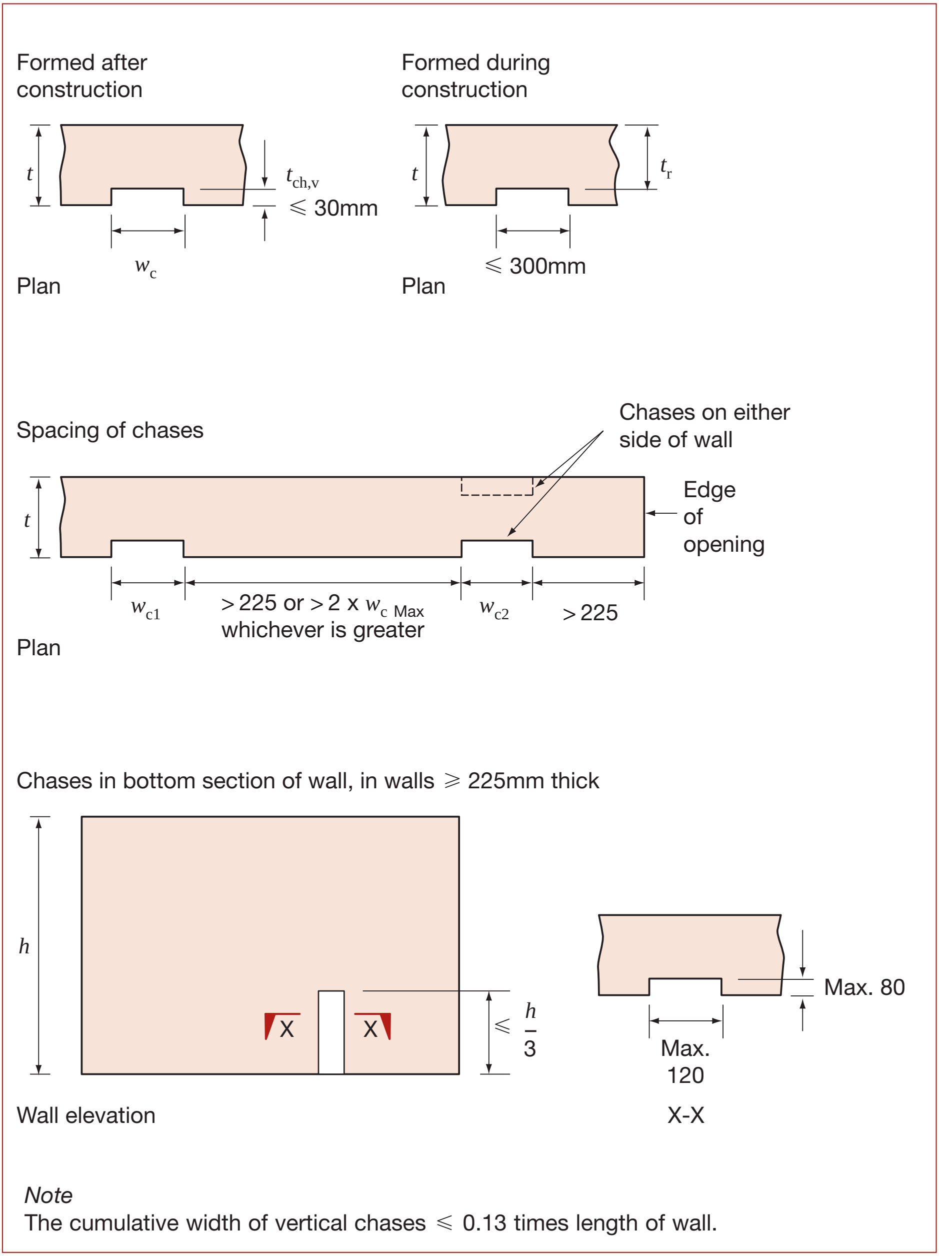


Fig 6.2 Vertical chases in loadbearing masonry walls – limitations (read with Table 6.1)

Table 6.2 Sizes of horizontal and inclined chases in masonry, allowed without calculation

Thickness of wall t (mm)	Maximum depth $t_{ch,h}$ (mm)	
	Unlimited length l_{ch}	Length $l_{ch} \leqslant$ 1250mm
75 – 84	0	0
85 – 115	0	0
116 – 175	0	15
176 – 225	10	20
226 – 300	15	25
over 300	20	30

Notes

a The maximum depth of the chase should include the depth of any hole reached when forming the chase.

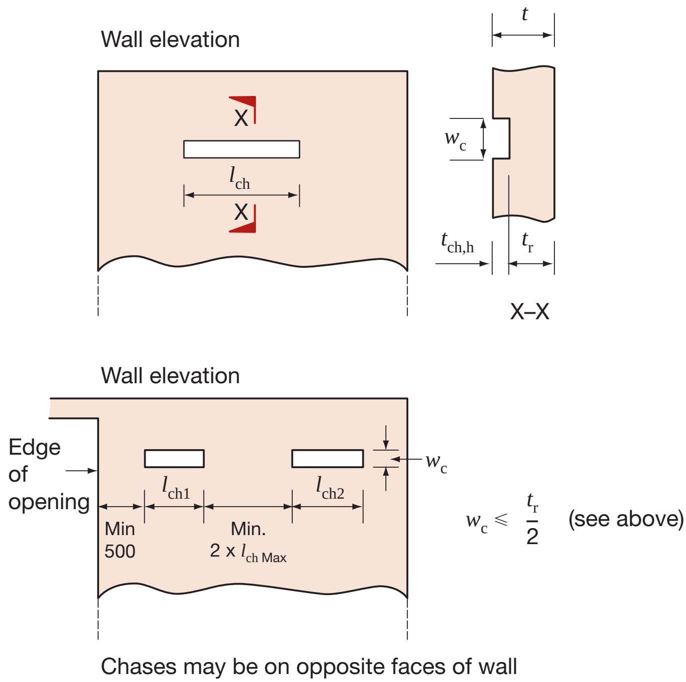
b The horizontal distance between the end of a chase and an opening should not be less than 500mm.

c The horizontal distance between adjacent chases of limited length, whether they occur on the same side or on opposite sides of the wall, should be not less than twice the length of the longest chase.

d In walls of thickness greater than 115mm, the permitted depth of the chase may be increased by 10mm if the chase is machine cut accurately to the required depth. If machine cuts are used, chases up to 10mm deep may be cut in both sides of walls of thickness not less than 225mm.

e The width of chase should not exceed the residual thickness of the wall.

f This table is based on data from NA[4] to EC6 Part 1-1.



Note

a For walls thicker than 175mm, $t_{ch,h}$ may be increased by 10mm if accurate machine cutting is used.

b Horizontal chases should be positioned within one eighth of the clear height of the wall (above or below a floor).

c The rules for horizontal chases also apply to inclined chases.

Fig 6.3 Horizontal and inclined chases in loadbearing masonry walls – limitations (read with Table 6.2)

6.5 Damp proof courses

Careful consideration is required so that the details of both the horizontal and vertical damp proof courses and cavity trays are practical, provide a continuous barrier to the passage (ingress) of water, and can be constructed on site. Changes in direction of damp proof courses whether horizontal or vertical, and the junctions between horizontal and vertical damp proof courses, may, if not properly designed or considered, direct water into the building. In the case of cavity trays, lack of support to the damp proof course over the cavity may also lead to water ingress.

Horizontal, flexible damp proof courses should be sandwiched in a mortar bed and two courses of brick sized units or one course of block sized units built immediately so that there is minimum disturbance of the mortar joint between the units and the damp proof course until the mortar has set. Clay unit damp proof courses should be constructed in the same way as normal masonry, not less than two courses high, with staggered cross-joints and laid in an M12 mortar.

Flexible damp proof courses and cavity trays should be laid, preferably, such that they just project outside the face of the wall. In cavity construction every third or fourth perpend joint in the external leaf should be left unfilled in the masonry course immediately above the damp proof course to allow any water retained by the damp proof course or cavity tray to drain out.

Damp proof courses, whether flexible or rigid, should not be pointed or rendered over since this will allow water to bridge the damp proof course.

Horizontal damp proof courses should be capable of transferring the design loads applied to them (shear and vertical) without suffering or causing damage, and have sufficient frictional resistance to prevent movement of the masonry.

Where it is necessary to transmit tension across a dpc, clay dpc units should be used, see Section 3.3. Flexible dpcs cannot be relied upon for this purpose.

6.6 Overhangs, corbels and cornices

Experience shows that these details are often expensive and difficult to construct. Simple details are preferable. It is important to consider construction tolerances when formulating details. Details using special stainless steel fabrication to achieve the desired effect, and provide adequate tolerance in all directions, are usually expensive. Such details may introduce significant eccentricity and moments into the masonry elements to which they are attached and require consideration during design.

6.7 Thermal and long term movement

Masonry will expand and contract as a result of reversible thermal and moisture changes, and also as a result of irreversible moisture changes. These movements must be allowed for in design and detailing of the masonry. This is usually done by the provision of movement joints, both horizontal and vertical in the appropriate locations, see Section 2.8. Other movement may result from factors such as deflection from load or creep and as a result of ground movement.

6.8 Tolerances

Most contemporary buildings are composed of a mixture of factory-made components and onsite construction and, therefore, generally have a mix of significantly different orders of dimensional accuracy. Any building or component manufacturing process, whatever the material, produces work that falls short of perfect accuracy. Dimensional variability is, therefore, inherent in building materials and processes and is characteristic of them, and should be allowed for in the details.

In the case of infill masonry to framed construction, the frame, at any level, will normally be completed before masonry construction begins. In this instance, provided that the spacing between the vertical and horizontal frame members is based on the masonry unit dimensions, any inaccuracies in the frame dimensions can usually be allowed for by adjustment of the mortar joints, bed or perpends of the masonry. However, in the case of vertical alignment between storeys, or horizontal alignment between successive bays, the inaccuracies in the construction of the frame can be allowed for in the masonry only by the provision of adequate ‘play’ in the details, i.e. the finished masonry should be true to line and plumb, irrespective of the inaccuracies in the frame.

In loadbearing masonry construction the problems of dimensional inaccuracy are usually in the fit of items installed later, e.g. doors, windows, floors and roofs, and cladding. This is true of masonry cladding that is not constructed at the same time as the masonry frame.

In the case of doors and windows, these can be built in at the time of constructing the walls, but this could result in unacceptable subsequent damage to the door or window framing. It is common practice, therefore, to leave oversize holes into which the frame can be inserted. However, to allow for both under- and over-sizing of the frame and out-of-squareness of the formed hole, the tolerances to be detailed need to be reasonably generous. Temporary framing templates to form openings are an alternative option.

In the instance of in-situ floors or roofs any inaccuracies in the masonry can usually be taken up in the cutting, and fitting of the floor or roof construction. Where the floors or roof members are made offsite the tolerances should take account of the minimum bearings of the floor or roof and the details arranged to accommodate any possible over-sailing at the bearings.

EC6 Part 2[3] gives the limits on deviations in loadbearing masonry as shown in Table 6.3 and Figure 6.4 within which the formulae in the code are considered to be valid. Construction tolerances should be specified by the designer but should not exceed these values.

6.9 Inspections and acceptance

The quality of the construction is the contractor’s responsibility, and examination of the work by the designers or their site representatives should involve visits only to check that the requirements of the drawings and specification are satisfied.

The specification should cover tests of the quality of materials and mortar-mixing so that site inspections essentially examine the standard of workmanship.

The reduction in strength from unfilled perpend joints is mainly of significance in laterally loaded masonry. Unfilled perpend joints may also have adverse implications for fire resistance and sound insulation.

Inadequate standards of workmanship will also reduce the resistance of masonry to rain penetration and frost action and increase the requirements for maintenance because of reduced durability.

6.10 Partition walls

Non-loadbearing partition walls are not specifically excluded by EC6. It is, of course, possible to design such walls in accordance with EC6 allowing for wind or impact loading, as considered appropriate. This is rarely necessary, although guidance is desirable to ensure that excessively high, long or slender partitions are not constructed. Guidance is provided in BS 5628 Part 3[8] for the sizing of such walls and is included in this *Manual* as Figure 6.5.

Table 6.3 Permissible deviations for masonry elements

Position	Maximum deviation (mm)
Verticality:	
in any one storey	± 20mm
in total height of building of three storeys or more	± 50mm
vertical alignment	± 20mm
Straightness[a]:	
in any one metre	± 5mm
in 10 metres	± 50mm
Thickness:	
of wall leaf[b]	± 5mm or ± 5% of the leaf thickness whichever is the greater
of overall cavity wall	± 10mm

Notes

a Deviation from straightness is measured from a straight reference line between any two points.

b Excluding leaves of single masonry unit width or length, where the dimensional tolerances of the masonry units govern the leaf thickness.

c The deviations given in this table are not intended to be used as construction tolerances.

d This table is based on data from EC6 Part 2[3].

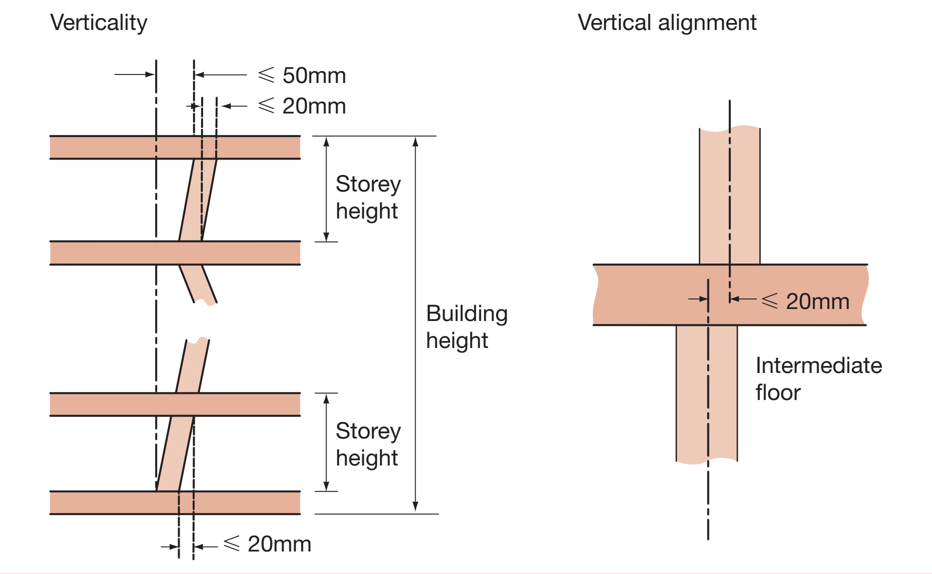
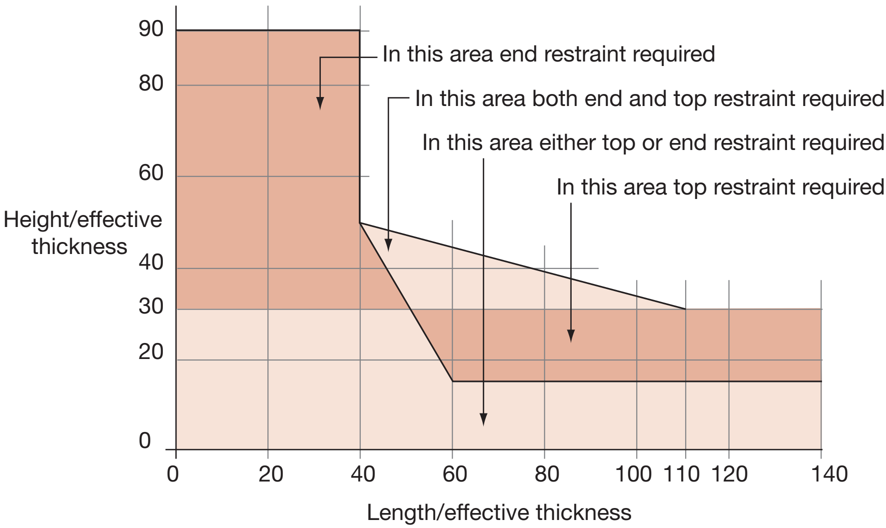


Fig 6.4 Maximum vertical deviations within which the formulae in the code are considered to be valid



Note
Outside the tinted areas, walls are unstable.

Fig 6.5 Limiting dimensions of walls for stability

7.1 General

The general objectives of fire protection are to limit risks with respect to the individual and society, neighbouring property and, where required, directly exposed property, in the case of fire.

The Construction Products Directive 89/106/EEC[61] gives the following essential requirement for the limitation of fire risks:

'The construction works must be designed and built in such a way that, in the event of an outbreak of fire:

- the loadbearing resistance of the construction can be assumed for a specified period of time
- the generation and spread of fire and smoke within the works are limited
- the occupants can leave the works or can be rescued by other means
- the safety of rescue teams is taken into consideration.'

The essential requirement may be observed by following various possibilities for fire safety strategies such as conventional fire scenarios (nominal fires) or 'natural' (parametric) fire scenarios, including passive and/or active fire protection measures.

BS EN 1996-1-2[2] deals with specific aspects of passive fire protection in terms of designing structures and parts thereof for adequate loadbearing and non-loadbearing walling resistance that could be needed for safe evacuation of occupants and fire rescue operations and for limiting fire spread as relevant.

Required functions and levels of performance are generally specified in terms of a standard fire resistance rating. Where fire safety engineering for assessing passive and active measures is acceptable, requirements will be less prescriptive and may allow for alternative strategies.
Supplementary requirements concerning, for example:

- the possible installation and maintenance of sprinkler systems
- conditions of occupancy of building or fire compartment
- the use of approved insulation and coating materials, including their maintenance,

are not given in BS EN 1996-1-2[2], as they are subject to specification by the competent authority.

A full analytical procedure for structural fire design would take into account the behaviour of the structural system at elevated temperatures, the potential heat exposure and the beneficial effects of active fire protection systems, together with the uncertainties associated with these three features and the importance of the structure (consequences of failure).

At the present time it is theoretically possible to perform a calculation procedure for determining adequate performance which incorporates some, if not all, of these parameters and to demonstrate that the structure, or its components, will give adequate performance in a real building fire. However, because of a lack of data, the principal current procedure in European countries is one based on results from standard fire tests. The grading system in regulations, which call for specific periods of fire resistance, takes into account (though not explicitly) the features and uncertainties described above.

Application of Part 1-2[2] of EC6 with the thermal actions given in EC1 Part 1-2[62] is illustrated in Figure 7.1. For design in accordance with EC6 Part 1-2[2], EC1 Part 1-2[62] is required for the determination of temperature fields in structural elements, or when using general calculation models for the analysis of the structural response.

The Institution of Structural Engineers' publications[63,64] give guidance on fire safety engineering of structures.

7.2 Scope

Part 1-2 of EC6[2] deals with the design of masonry structures for the accidental exposure of those structures to fire. It is intended to be used with Parts 1-1[1], 2[3] and 3[9] of EC6 and Part 1-2 of EC1[62] (the Eurocode for Actions on Structures). Part 1-2[2] of EC6 identifies differences from normal temperature design or supplements that design.

Part 1-2[2] deals only with passive methods of fire protection; active methods are not covered. It is to be used where masonry structures are required to avoid premature collapse in fires and where it is necessary to limit fire spread beyond designated areas. Principles and application rules are given to enable masonry structures to fulfil these requirements.

Part 1-2[2] applies to non-loadbearing and to loadbearing internal and external masonry walls which fulfil both separating and non-separating functions. It does not deal with natural stone masonry built with units complying with BS EN 771-6[23]. In the UK we are interested in separating walls only and the National Annex to EC6 Part 1-2[5] gives tabulated data accordingly.

7.3 National Annex for BS EN 1996-1-2

EC6 Part 1-2[2] is used in the UK with its National Annex[5]. National choice is allowed in Part 1-2 in the Clauses below.

- 2.2(2) Actions
- 2.3(2) Design values of material properties
- 2.4.2(3) Member analysis
- 3.3.3.1(1) Thermal elongation
- 3.3.3.2(1) Specific heat capacity
- 3.3.3.3 Thermal conductivity
- 4.5(3) Assessment by tabulated data
- Annex B Values of t_F and l_F.

Because the calculation methods given in Annexes C and D of EC6 Part 1-2[2] will not be used in the UK, no values are given for any of the symbols in Clauses 2.2(2) to 4.5(3) above in the National Annex[5]. The National Annex[5] only gives guidance on the use of the EC6 Part 1-2 Annex B[2] fire tables which are based on the national fire database. Annexes A and E can also be used.

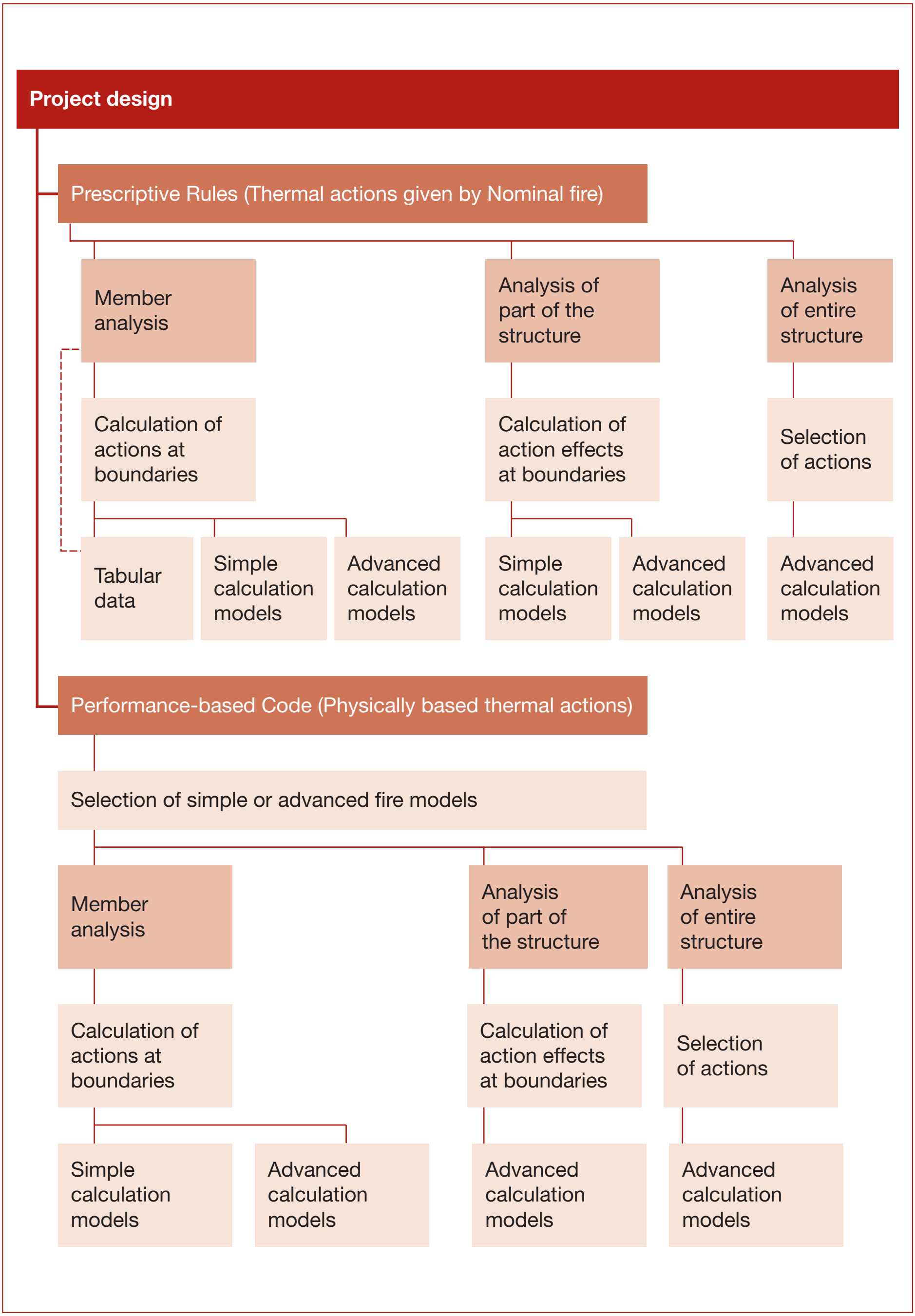


Fig 7.1 Design procedures

7.4 Principles

EC6 Part 1-2[2] requires consideration of masonry elements in terms of function, i.e. loadbearing, separating and mechanical impact resisting, and combinations thereof.

The analysis of masonry in a fire situation may be carried out by:

- testing the structure
- tabulated data
- member analysis
- analysis of part of the structure
- global structural analysis.

Part 1-2[2] gives tabulated data based on the standard temperature-time curve in accordance with BS EN 1363[65].

7.5 Materials

The requirements for masonry units and mortars in EC6 Part 1-2[2] are the same as those given in EC6 Part 1-1[1], with the addition of a further group of units for clay and calcium silicate, reference 1S, which includes units containing less than 5% of formed voids. These units may have indentations (e.g. frogs), providing that the indentations are filled with mortar in the finished wall.

The properties of these materials, as given in EC6 Part 1-1[1], are considered to apply at a temperature of 20ºC. EC6 Part 1-2[2] contains stress-strain relationships at elevated temperatures for some materials.

7.6 Design procedures

Three methods for assessing the fire resistance of masonry walls are considered in EC6 Part 1-2[2], namely, by testing, by the use of tabulated data and by calculation. The UK National Annex[5], however, does not support the use of calculation methods.

Testing is required where no data is available for a particular combination of masonry unit and mortar. Interpretation of the test results should allow for any differences in the test from the codified test methods.

In assessing the fire resistance of a masonry wall using the tabulated data given in EC6 Part 1-2[2], it is necessary to know the details of the masonry units being used (density etc.), the grouping into which the units fall (see Table 3.1 in EC6 Part 1-1[1]), the type of mortar used and any applied finish. The form and function of the wall is then considered (e.g. separating construction in single leaf, loadbearing, cavity etc.), see Section 2.1.2 of EC6 Part 1-2[2].

Masonry members must be considered against various criteria in relation to their fire resistance for standard fire exposure. These are:

- R – mechanical resistance
- E – integrity
- I – insulation
- M – mechanical impact.

The form and function of masonry walls in relation to nominal fire exposure are defined by these criteria, thus:

- loadbearing only – criterion R
- separating only – criteria EI
- separating and loadbearing – criteria REI
- loadbearing, separating and mechanical impact – criteria REI-M
- separating and mechanical impact – criteria EI-M.

Note that 'mechanical impact' is not considered to be relevant in the UK so the National Annex[5] does not contain tabulated data for the last two criteria.

If the element is loadbearing, it is necessary to verify its loadbearing capacity in accordance with EC6 Part 1-1[1] and to carry out an additional check on the slenderness ratio of the wall. Having verified the loadbearing adequacy of the wall, the minimum thickness for the relevant criterion, to achieve the stated period of fire resistance for the materials used, can be obtained from the Tables in Annex B to EC6 Part 1-2[2]. In order to use the tables for loadbearing walls in the National Annex[5] the level of loading, as obtained in the capacity calculation, is required, as well as the masonry materials being used. The design load is compared with the design resistance of the wall in order to ascertain the percentage loading. If the percentage is greater than 60%, then the tabulated values for 100% loading are used. If the percentage is 60% or less, then the 60% loading tabulated values are used.

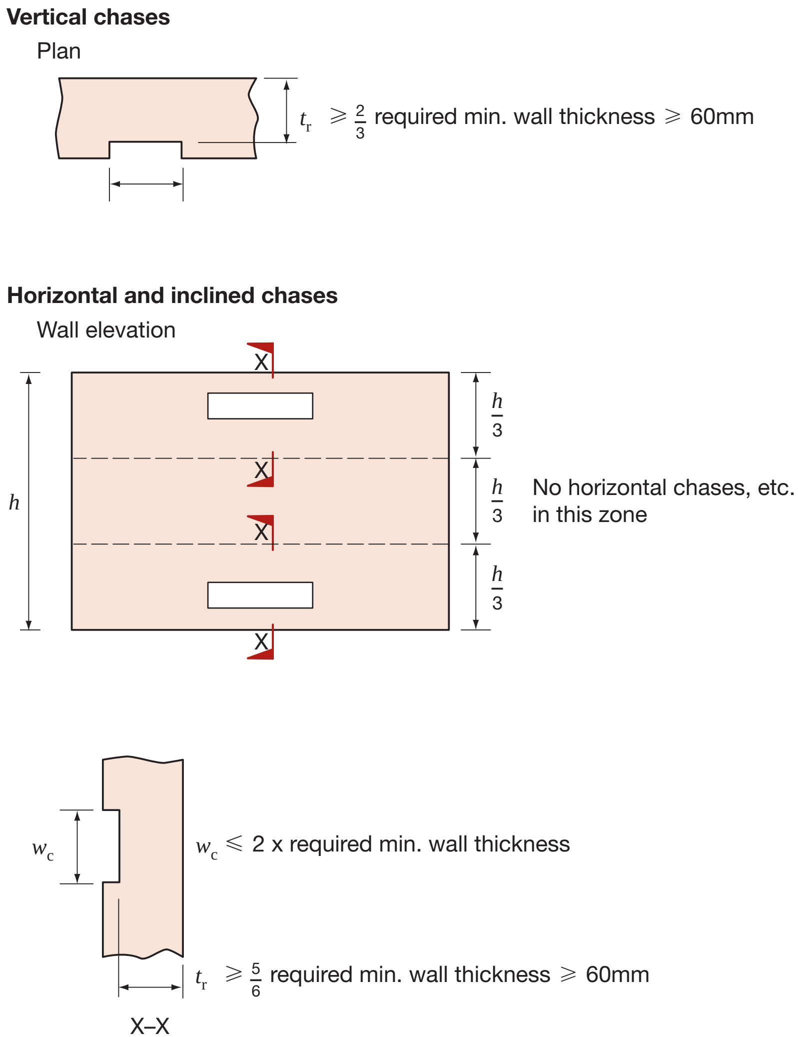
Tables 7.1 to 7.14 in Section 7.8 give the minimum thickness of the wall or leaf exposed to fire for specific fire resistance periods related to masonry materials and the criteria given above.

7.7 Detailing

Patently, the detailing of joints, connections, holes for service penetrations, chases and recesses, etc. is of considerable importance in relation to the fire resistance of masonry construction. EC6 Part 1-2[2] makes it clear that either these items should not reduce the fire resistance of the construction or they should be taken into account in assessing the fire resistance. Thus, joints in walls and between walls must be appropriately designed and constructed. Any fire insulating layers in movement joints must have a melting point of at least 1000ºC and be tightly sealed so that movement does not affect the fire resistance. Connections which are between fire walls and concrete or masonry structures which are required to satisfy mechanical impact requirements must be in joints that are completely filled with mortar or concrete or properly protected against fire.

Chases and recesses in loadbearing walls which satisfy the requirements of EC6 Part 1-1[1] (see Section 6.4 in this *Manual*) can be assumed to have no adverse effect on the fire resistance of the wall. In non-loadbearing walls, vertical chases and recesses should leave at least 2/3 of the wall intact, but not less than 60mm. Horizontal and inclined chases and recesses should leave 5/6 of the wall intact, but not less than 60mm. Horizontal and inclined chases and recesses should not be located in the middle one-third height of the wall, and the width of individual chases and recesses should not be greater than twice the required minimum thickness of the wall. These provisions are illustrated in Figure 7.2.

Individual cables can pass through holes sealed with mortar and non-combustible pipes up to 100mm diameter may pass through non-combustible sealed holes. Other service penetrations must be sealed by a method which has been evaluated by test or experience to be satisfactory.



Note
The required min. wall thickness includes any integrally applied fire resistant finish such as plaster.

Fig 7.2 Chases and recesses in fire resisting non-loadbearing walls

7.8 Tabulated fire resistance of masonry walls

The following tables (7.1 to 7.14) give the minimum thickness of loadbearing masonry walls for specific fire resistance periods, from 30 to 240 minutes, for various masonry and wall types. The tables are only valid for walls complying with EC6 Part 1-1[1], Part 2[3] and Part 3[9]. They are based on the values given in the NA to EC6 Part 1-2[5]. The thicknesses given in the tables are for the masonry alone, excluding finishes. For each specification, the top row of figures in each table is for walls without an applied finish or just a thin rendering. The values in brackets are for walls having an applied finish of gypsum premixed plaster to BS EN 13279-1[66] or plaster type LW or T in accordance with BS EN 998-1[67]. The plaster is to be at least 10mm thick on both faces of a single leaf wall or on the fire exposed face of a cavity wall. Sand cement render is not considered to increase the fire resistance of the wall. For high precision units or tongued and grooved units with narrow, unfilled perpend joints, reference should be made to the code. Walls which include bed joint reinforcement to BS EN 845-3[35] are covered by the tables.

The tables applicable to $\alpha \leqslant 0.6$ may be used when the verification of vertical load capacity indicates that only up to 0.6 of the permitted design vertical load resistance is being used. $\alpha \leqslant 1.0$ should be used when more than 0.6 of the permitted capacity is being used.

Table 7.1 Clay masonry: minimum thickness of separating non-loadbearing walls (criteria EI) for fire resistance classifications

Material properties: gross density ρ (kg/m³)	**Minimum wall thickness t_F (mm) for fire resistance classification EI for time $t_{fi,d}$ (mins) of:**					
	30	60	90	120	180	240
Group 1S units						
Mortar: general purpose, thin layer lightweight						
$\rho \geqslant 1200$	**65**	**65**	**90**	**100**	**170**	**170**
	(65)	**(65)**	**(90)**	**(100)**	**(100)**	**(140)**
Group 1 units						
Mortar: general purpose, thin layer lightweight						
$\rho \geqslant 1000$	**65**	**100**	**100**	**100**	**170**	**200**
	(65)	**(65)**	**(90)**	**(100)**	**(140)**	**(170)**
Group 2 units						
Mortar: general purpose, thin layer lightweight						
$\rho \geqslant 700$ 25% < perforation $\leqslant$ 40%	**100**	**130**	**215**	**215**	**240**	**240**
	(100)	**(130)**	**(215)**	**(215)**	**(215)**	**(240)**
Note This table is based on data from NA to EC6 Part 1-2[5].						

Table 7.2 Clay masonry: minimum thickness of separating loadbearing single-leaf walls (criteria REI) for fire resistance classifications

Material properties: gross density ρ (kg/m³)		Minimum wall thickness t_F (mm) for fire resistance classification REI for time $t_{fi,d}$ (mins) of:					
		30	60	90	120	180	240
Group 1S units							
Mortar: general purpose, thin layer							
$\rho \geqslant 1200$	$\alpha \leqslant 1.0$	**90**	**90**	**100**	**100**	**170**	**170**
		(90)	**(90)**	**(90)**	**(100)**	**(140)**	**(140)**
	$\alpha \leqslant 0.6$	**90**	**90**	**100**	**100**	**170**	**170**
		(90)	**(90)**	**(90)**	**(100)**	**(100)**	**(140)**
Group 1 units							
Mortar: general purpose, thin layer							
$\rho \geqslant 1000$	$\alpha \leqslant 1.0$	**100**	**100**	**100**	**140**	**200**	**200**
		(90)	**(100)**	**(100)**	**(100)**	**(170)**	**(170)**
	$\alpha \leqslant 0.6$	**90**	**100**	**100**	**140**	**170**	**200**
		(90)	**(90)**	**(100)**	**(100)**	**(140)**	**(170)**
Group 2 units							
Mortar: general purpose, thin layer							
$\rho \geqslant 700$ 25% < perforation $\leqslant$ 40%	$\alpha \leqslant 1.0$	**100**	**130**	**215**	**215**	**240**	**240**
		(100)	**(130)**	**(215)**	**(215)**	**(215)**	**(240)**
	$\alpha \leqslant 0.6$	**100**	**130**	**215**	**215**	**240**	**240**
		(100)	**(130)**	**(215)**	**(215)**	**(215)**	**(240)**

Note

This table is based on data from NA to EC6 Part 1-2[5].

Table 7.3 Clay masonry: minimum thickness of each leaf of separating loadbearing cavity walls with one leaf loaded (criteria REI) for fire resistance classifications

Material properties: gross density ρ (kg/m³)		**Minimum wall thickness t_F (mm) for fire resistance classification REI for time $t_{fi,d}$ (mins) of:**					
		30	60	90	120	180	240
Group 1S and Group 1 units							
Mortar: general purpose, thin layer							
$\rho \geqslant 1000$	$\alpha \leqslant 1.0$	**90**	**90**	**100**	**100**	**170**	**170**
		(90)	**(90)**	**(90)**	**(100)**	**(140)**	**(140)**
	$\alpha \leqslant 0.6$	**90**	**90**	**100**	**100**	**170**	**170**
		(90)	**(90)**	**(90)**	**(100)**	**(140)**	**(140)**
Group 2 units							
Mortar: general purpose, thin layer							
$\rho \geqslant 700$ 25% < perforation $\leqslant$ 40%	$\alpha \leqslant 1.0$	**100**	**130**	**215**	**215**	**240**	**240**
		(100)	**(130)**	**(215)**	**(215)**	**(240)**	**(240)**
	$\alpha \leqslant 0.6$	**100**	**130**	**215**	**215**	**240**	**240**
		(100)	**(130)**	**(215)**	**(215)**	**(240)**	**(240)**

Notes

a The tabulated thicknesses are for the loaded leaves of cavity walls where the loaded leaf is subjected to fire.

b The non-loaded leaf may be of a dissimilar material to the loaded leaf, but should otherwise conform to the relevant material specifications. In such cases, the respective thicknesses of each leaf should conform to that specified in the appropriate material table.

c This table is based on data from NA to EC6 Part 1-2[5].

Table 7.4 Calcium silicate masonry: minimum thickness of separating non-loadbearing walls (criteria EI) for fire resistance classifications

Material properties: gross density ρ (kg/m³)	**Minimum wall thickness t_F (mm) for fire resistance classification EI for time $t_{fi,d}$ (mins) of:**					
	30	60	90	120	180	240
Group 1S units						
Mortar: general purpose						
$\rho \geqslant 1600$	**65**	**65**	**90**	**100**	**170**	**170**
	(65)	**(65)**	**(90)**	**(100)**	**(100)**	**(140)**
Group 1 units						
Mortar: general purpose, thin layer						
$\rho \geqslant 1000$	**65**	**100**	**100**	**100**	**170**	**200**
	(65)	**(65)**	**(90)**	**(100)**	**(140)**	**(170)**

Note

This table is based on data from NA to EC6 Part 1-2[5].

Table 7.5 Calcium silicate masonry: minimum thickness of separating loadbearing single-leaf walls (criteria REI) for fire resistance classifications

Material properties: gross density ρ (kg/m³)		Minimum wall thickness t_F (mm) for fire resistance classification REI for time $t_{fi,d}$ (mins) of:					
		30	60	90	120	180	240
Group 1S units							
Mortar: general purpose, thin layer							
$\rho \geqslant 1600$	$\alpha \leqslant 1.0$	**90**	**90**	**100**	**100**	**190**	**190**
		(90)	**(90)**	**(90)**	**(100)**	**(140)**	**(190)**
	$\alpha \leqslant 0.6$	**90**	**90**	**100**	**100**	**170**	**190**
		(90)	**(90)**	**(90)**	**(100)**	**(100)**	**(190)**
Group 1 units							
Mortar: general purpose, thin layer							
$\rho \geqslant 1000$	$\alpha \leqslant 1.0$	**100**	**100**	**100**	**190**	**200**	**200**
		(90)	**(100)**	**(100)**	**(100)**	**(170)**	**(170)**
	$\alpha \leqslant 0.6$	**90**	**100**	**100**	**170**	**190**	**200**
		(90)	**(90)**	**(100)**	**(100)**	**(170)**	**(170)**

Note

This table is based on data from NA to EC6 Part 1-2[5].

Table 7.6 Calcium silicate masonry: minimum thickness of each leaf of separating loadbearing cavity walls with one leaf loaded (criteria REI) for fire resistance classifications

Material properties: gross density ρ (kg/m³)		Minimum wall thickness t_F (mm) for fire resistance classification REI for time $t_{fi,d}$ (mins) of:					
		30	60	90	120	180	240
Group 1S and Group 1 units							
Mortar: general purpose, thin layer							
$\rho \geqslant 1000$	$\alpha \leqslant 1.0$	**90**	**90**	**100**	**100**	**170**	**170**
		(90)	**(90)**	**(90)**	**(100)**	**(140)**	**(140)**
	$\alpha \leqslant 0.6$	**90**	**90**	**100**	**100**	**170**	**170**
		(90)	**(90)**	**(90)**	**(100)**	**(140)**	**(140)**

Notes

a The tabulated thicknesses are for the loaded leaves of cavity walls where the loaded leaf is subjected to fire.

b The non-loaded leaf may be of a dissimilar material to the loaded leaf, but should otherwise conform to the relevant material specifications. In such cases, the respective thicknesses of each leaf should conform to that specified in the appropriate material table.

c This table is based on data from NA to EC6 Part 1-2[5].

Table 7.7 Dense and lightweight aggregate concrete masonry: minimum thickness of separating non-loadbearing separating walls (criteria EI) for fire resistance classifications

Material properties: gross density ρ (kg/m³)	Minimum wall thickness t_F (mm) for fire resistance classification EI for time $t_{fi,d}$ (mins) of:					
	30	60	90	120	180	240
Group 1 units						
Mortar: general purpose, thin layer, lightweight						
Lightweight aggregate $400 \leqslant \rho \leqslant 1700$	**50**	**70**	**75**	**75**	**90**	**100**
	(50)	**(50)**	**(60)**	**(70)**	**(75)**	**(75)**
Dense aggregate $1200 \leqslant \rho \leqslant 2400$	**50**	**70**	**90**	**90**	**100**	**100**
	(50)	**(50)**	**(70)**	**(75)**	**(90)**	**(100)**
Group 2 units						
Mortar: general purpose, thin layer, lightweight						
Lightweight aggregate $240 \leqslant \rho \leqslant 1300$	**50**	**70**	**75**	**100**	**115**	**125**
	(50)	**(50)**	**(70)**	**(75)**	**(90)**	**(100)**
Dense aggregate $720 \leqslant \rho \leqslant 1800$	**90**	**100**	**125**	**140**	**140**	**140**
	(70)	**(80)**	**(90)**	**(100)**	**(125)**	**(125)**

Note
This table is based on data from NA to EC6 Part 1-2[5].

Table 7.8 Dense and lightweight aggregate concrete masonry: minimum thickness of separating loadbearing single-leaf walls (criteria REI) for fire resistance classifications

Material properties: gross density ρ (kg/m^3)		**Minimum wall thickness t_F (mm) for fire resistance classification REI for time $t_{fi,d}$ (mins) of:**					
		30	60	90	120	180	240
Group 1 units							
Mortar: general purpose, thin layer, lightweight							
Lightweight aggregate $400 \leqslant \rho \leqslant 1700$	$\alpha \leqslant 1.0$	**90**	**90**	**100**	**100**	**140**	**150**
		(90)	**(90)**	**(90)**	**(90)**	**(100)**	**(100)**
	$\alpha \leqslant 0.6$	**70**	**75**	**90**	**90**	**100**	**100**
		(60)	**(60)**	**(75)**	**(75)**	**(90)**	**(90)**
Dense aggregate $1200 \leqslant \rho \leqslant 2400$	$\alpha \leqslant 1.0$	**90**	**90**	**90**	**100**	**140**	**150**
		(90)	**(90)**	**(90)**	**(90)**	**(100)**	**(100)**
	$\alpha \leqslant 0.6$	**75**	**75**	**90**	**90**	**100**	**140**
		(60)	**(75)**	**(75)**	**(75)**	**(90)**	**(100)**
Group 2 units							
Mortar: general purpose, thin layer, lightweight							
Lightweight aggregate $240 \leqslant \rho \leqslant 1300$	$\alpha \leqslant 1.0$	**90**	**100**	**100**	**100**	**140**	**150**
		(90)	**(90)**	**(90)**	**(100)**	**(140)**	**(140)**
	$\alpha \leqslant 0.6$	**75**	**90**	**90**	**100**	**125**	**140**
		(75)	**(75)**	**(75)**	**(90)**	**(100)**	**(125)**
Dense aggregate $720 \leqslant \rho \leqslant 1800$	$\alpha \leqslant 1.0$	**100**	**100**	**140**	**140**	**140**	**190**
		(90)	**(100)**	**(100)**	**(140)**	**(140)**	**(150)**
	$\alpha \leqslant 0.6$	**90**	**100**	**100**	**140**	**140**	**150**
		(75)	**(90)**	**(90)**	**(125)**	**(125)**	**(140)**

Note

This table is based on data from NA to EC6 Part 1-2[5].

Table 7.9 Dense and lightweight aggregate concrete masonry: minimum thickness of each leaf of separating loadbearing cavity walls with one leaf loaded (criteria REI) for fire resistance classifications

Material properties: gross density ρ (kg/m³)		Minimum wall thickness t_F (mm) for fire resistance classification REI for time $t_{fi,d}$ (mins) of:					
		30	60	90	120	180	240
Group 1 units							
Mortar: general purpose, thin layer, lightweight							
Lightweight aggregate $400 \leqslant \rho \leqslant 1700$	$\alpha \leqslant 1.0$	**90**	**90**	**100**	**100**	**140**	**150**
		(90)	**(90)**	**(90)**	**(100)**	**(100)**	**(100)**
	$\alpha \leqslant 0.6$	**70**	**75**	**90**	**90**	**100**	**100**
		(60)	**(60)**	**(75)**	**(75)**	**(90)**	**(90)**
Dense aggregate $1200 \leqslant \rho \leqslant 2400$	$\alpha \leqslant 1.0$	**90**	**90**	**100**	**100**	**140**	**150**
		(90)	**(90)**	**(90)**	**(90)**	**(100)**	**(100)**
	$\alpha \leqslant 0.6$	**75**	**75**	**90**	**90**	**100**	**140**
		(60)	**(75)**	**(75)**	**(75)**	**(90)**	**(125)**
Group 2 units							
Mortar: general purpose, thin layer, lightweight							
Lightweight aggregate $240 \leqslant \rho \leqslant 1300$	$\alpha \leqslant 1.0$	**90**	**100**	**100**	**100**	**140**	**150**
		(90)	**(90)**	**(90)**	**(100)**	**(140)**	**(140)**
	$\alpha \leqslant 0.6$	**70**	**90**	**90**	**100**	**125**	**140**
		(70)	**(70)**	**(70)**	**(90)**	**(100)**	**(125)**
Dense aggregate $720 \leqslant \rho \leqslant 1800$	$\alpha \leqslant 1.0$	**90**	**100**	**100**	**100**	**140**	**190**
		(90)	**(90)**	**(100)**	**(100)**	**(140)**	**(150)**
	$\alpha \leqslant 0.6$	**90**	**100**	**100**	**100**	**140**	**150**
		(70)	**(90)**	**(90)**	**(100)**	**(125)**	**(140)**

Notes

a The tabulated thicknesses are for the loaded leaves of cavity walls where the loaded leaf is subjected to fire.

b The non-loaded leaf may be of a dissimilar material to the loaded leaf, but should otherwise conform to the relevant material specifications. In such cases, the respective thickness of each leaf should conform to that specified in the appropriate material table.

c This table is based on data from NA to EC6 Part 1-2[5].

Table 7.10 Autoclaved aerated concrete masonry: minimum thickness of separating non-loadbearing walls (criteria EI) for fire resistance classifications

Material properties: gross density ρ (kg/m³)	Minimum wall thickness t_F (mm) for fire resistance classification EI for time $t_{fi,d}$ (mins) of:					
	30	60	90	120	180	240
Group 1 and 1S units						
Mortar: general purpose, thin layer						
$350 \leqslant \rho \leqslant 500$	**65**	**65**	**70**	**70**	**100**	**100**
	(50)	**(65)**	**(70)**	**(70)**	**(100)**	**(100)**
$500 \leqslant \rho \leqslant 1000$	**50**	**60**	**60**	**65**	**75**	**100**
	(50)	**(50)**	**(50)**	**(65)**	**(75)**	**(100)**

Note
This table is based on data from NA to EC6 Part 1-2[5].

Table 7.11 Autoclaved aerated concrete masonry: minimum thickness of separating loadbearing single-leaf walls (criteria REI) for fire resistance classifications

Material properties: gross density ρ (kg/m³)		Minimum wall thickness t_F (mm) for fire resistance classification REI for time $t_{fi,d}$ (mins) of:					
		30	60	90	120	180	240
Group 1 and 1S units							
Mortar: general purpose, thin layer							
$350 \leqslant \rho \leqslant 500$	$\alpha \leqslant 1.0$	**100**	**100**	**120**	**125**	**150**	**150**
		(90)	**(100)**	**(110)**	**(125)**	**(150)**	**(150)**
	$\alpha \leqslant 0.6$	**100**	**100**	**100**	**120**	**140**	**150**
		(90)	**(100)**	**(100)**	**(100)**	**(120)**	**(120)**
Mortar: general purpose, thin layer							
$500 \leqslant \rho \leqslant 1000$	$\alpha \leqslant 1.0$	**90**	**90**	**100**	**100**	**140**	**150**
		(90)	**(90)**	**(90)**	**(90)**	**(100)**	**(100)**
	$\alpha \leqslant 0.6$	**90**	**90**	**100**	**100**	**120**	**150**
		(90)	**(90)**	**(90)**	**(90)**	**(100)**	**(100)**

Note
This table is based on data from NA to EC6 Part 1-2[5].

Table 7.12 Autoclaved aerated concrete masonry: minimum thickness of each leaf of separating loadbearing cavity walls with one leaf loaded (criteria REI) for fire resistance classifications

Material properties: gross density ρ (kg/m³)		**Minimum wall thickness t_F (mm) for fire resistance classification REI for time $t_{fi,d}$ (mins) of:**					
		30	60	90	120	180	240
Group 1 and 1S units							
Mortar: general purpose, thin layer							
$350 \leqslant \rho \leqslant 500$	$\alpha \leqslant 1.0$	**90**	**90**	**100**	**100**	**150**	**150**
		(90)	**(90)**	**(100)**	**(100)**	**(150)**	**(150)**
	$\alpha \leqslant 0.6$	**90**	**90**	**90**	**100**	**150**	**150**
		(90)	**(90)**	**(90)**	**(100)**	**(150)**	**(150)**
Mortar: general purpose, thin layer							
$500 \leqslant \rho \leqslant 1000$	$\alpha \leqslant 1.0$	**90**	**90**	**100**	**100**	**140**	**150**
		(90)	**(90)**	**(100)**	**(100)**	**(140)**	**(150)**
	$\alpha \leqslant 0.6$	**90**	**90**	**100**	**100**	**125**	**150**
		(90)	**(90)**	**(100)**	**(100)**	**(125)**	**(150)**

Notes

a The tabulated thicknesses are for the loaded leaves of cavity walls where the loaded leaf is subjected to fire.

b The non-loaded leaf may be of a dissimilar material to the loaded leaf, but should otherwise conform to the relevant material specifications. In such cases, the respective thickness of each leaf should conform to that specified in the appropriate material table.

c This table is based on data from NA to EC6 Part 1-2[5].

Table 7.13 Manufactured stone masonry: minimum thickness of separating non-loadbearing separating walls (criteria EI) for fire resistance classifications

Material properties: gross density ρ (kg/m³)	Minimum wall thickness t_F (mm) for fire resistance classification EI for time $t_{fi,d}$ (mins) of:					
	30	60	90	120	180	240
Group 1 units						
Mortar: general purpose, thin layer, lightweight						
$1200 \leqslant \rho \leqslant 2400$	**50**	**70**	**90**	**90**	**100**	**100**
	(50)	**(50)**	**(70)**	**(75)**	**(90)**	**(100)**

Note

This table is based on data from NA to EC6 Part 1-2[5].

Table 7.14 Manufactured stone masonry: minimum thickness of separating loadbearing single-leaf walls (criteria REI) for fire resistance classifications

Material properties: gross density ρ (kg/m³)		Minimum wall thickness t_F (mm) for fire resistance classification REI for time $t_{fi,d}$ (mins) of:					
		30	60	90	120	180	240
Group 1 units							
Mortar: general purpose, thin layer, lightweight							
$1200 \leqslant \rho \leqslant 2400$	$\alpha \leqslant 1.0$	**90**	**90**	**90**	**100**	**140**	**150**
		(90)	**(90)**	**(90)**	**(90)**	**(100)**	**(100)**
	$\alpha \leqslant 0.6$	**75**	**75**	**90**	**90**	**100**	**140**
		(60)	**(75)**	**(75)**	**(75)**	**(90)**	**(100)**

Note

This table is based on data from NA to EC6 Part 1-2[5].

1 *BS EN 1996-1-1: 2005: Eurocode 6. Design of masonry structures. General rules for reinforced and unreinforced masonry structures*. London: BSI, 2005

2 *BS EN 1996-1-2: 2005: Eurocode 6. Design of masonry structures. General rules structural fire design*. London: BSI, 2005

3 *BS EN 1996-2: 2006: Eurocode 6. Design of masonry structures. Design considerations, selection of materials and execution of masonry*. London: BSI, 2006

4 *NA to BS EN 1996-1-1: 2005: UK National Annex to Eurocode 6. Design of Masonry Structures. General rules for reinforced and unreinforced masonry structures.* London: BSI, 2007

5 *NA to BS EN 1996-1-2: 2005: UK National Annex to Eurocode 6. Design of Masonry Structures. General rules for structural fire design.* London: BSI, 2007

6 *NA to BS EN 1996-2: 2006: UK National Annex to Eurocode 6. Design of masonry structures. Design considerations, selection of materials and execution of masonry.* London: BSI, 2007

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8 *BS 5628-3: 2005: Code of Practice for the use of masonry. Materials and components, design and workmanship.* London: BSI, 2005

9 *BS EN 1996-3: 2006: Eurocode 6. Design of masonry structures. Simplified calculation methods for unreinforced masonry structures*. London: BSI, 2006

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17 BRE. *Estimation of thermal and moisture movements and stresses. Part 3. BRE Digest 229.* Garston: BRE, 1979

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20 *BS EN 771-3: 2003: Specification for masonry units. Aggregate concrete masonry units (dense and light-weight aggregates).* London: BSI, 2004

21 *BS EN 771-4: 2003: Specification for masonry units. Autoclaved aerated concrete masonry units.* London: BSI, 2003

22 *BS EN 771-5: 2003: Specification for masonry units. Manufactured stone masonry units.* London: BSI, 2004

23 *BS EN 771-6: 2005: Specification for masonry units. Natural stone masonry units.* London: BSI, 2006

24 *BS EN 197-1: 2000: Cement composition, specifications and conformity criteria.* London: BSI, 2000

25 *BS 6699: 1992: Specification for ground granulated blast furnace slag for use with Portland cement.* London: BSI, 1992

26 *BS 7979: 2001: Specification for limestone fines for use with Portland cement.* London: BSI, 2001

27 *BS 3892-1: 1997: Pulverised-fuel ash.* London: BSI, 2007

28 *BS EN 450-1: 2005: Fly ash for concrete. Definition, specifications and conformity criteria.* London: BSI, 2005

29 *BS EN 413-1: 2004: Masonry Cement. Composition, specification and conformity criteria.* London: BSI, 2004

30 *PD 6678: 2005: Guide to the specification of masonry mortar.* London: BSI, 2005

31 *BS EN 998-2: 2003: Specification for mortar for masonry. Masonry mortar.* London: BSI, 2003

32 *BS EN 13139: 2002: Aggregates for mortar.* London: BSI, 2002

33 *BS EN 845-1: 2003: Specification for ancillary components for masonry. Ties, tension straps, hangers and brackets.* London: BSI, 2003

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44 *NA to BS EN 1991-1-3: 2003: UK National Annex to Eurocode 1: Actions on structures. General actions. Snow loads.* London: BSI, 2005

45 *BS EN 1991-1-4: 2005: Eurocode 1: Actions on structures. General actions. Wind loads.* London: BSI, 2005

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47 *BS EN 1991-1-7: 2006: Eurocode 1: Actions on structures. General actions. Accidental actions*. London: BSI, 2006

48 *NA to BS EN 1991-1-7: 2006: UK National Annex to Eurocode 1: Actions on structures. General actions. Accidental actions*. London: BSI, not yet published

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50 *NA to BS EN 1997-1: 2004: UK National Annex to Eurocode 7: Geotechnical design. General rules.* London: BSI, 2007

51 *BS EN 1990: 2002: Eurocode. Basis of structural design.* London: BSI, 2002

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53 *BS EN 1052-1: 1999: Methods of test for masonry. Determination of compressive strength.* London: BSI, 1999

54 *BS EN 772-1: 2000: Methods of test for masonry units. Determination of compressive strength.* London: BSI, 2000

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64 Institution of Structural Engineers. *Guide to the advanced fire safety engineering of structures.* London: IStructE, 2007

65 *BS EN 1363-1: 1999: Fire resistance tests. General requirements*. London: BSI, 1999

66 *BS EN 13279-1: 2005: Gypsum binders and gypsum plasters. Definitions and requirements.* London: BSI, 2005

67 *BS EN 998-1: 2003: Specification for mortar for masonry: rendering and plastering mortar.* London: BSI, 2003

68 *BS 5997-1: 1981: Lintels. Method for assessment of load.* London: BSI, 1981

Design data should include some or all of the following:

- company, contract, job number and date
- client, architect, engineer and checker responsible
- project organisation - details of any design subcontracts
- Building Regulation authority or other and date of submission
- general description of building, intended use, location, and any unusual environmental conditions
- consequence class
- site constraints
- principal design codes and other important reference documents used
- performance criteria
- design methodology including computer programs used
- materials and proprietary systems
- design assumptions
- structural form, stability and robustness provisions
- general loading conditions including environmental and exposure conditions
- fire resistance requirements
- durability
- soil and groundwater conditions (including contamination)
- foundation type and design
- drainage
- movement joints
- quality plan
- site supervisor's check list or critical structural information to be included in such a list
- health and safety including risk management and CDM
- maintenance recommendations
- other relevant data or information.

B.1 General

Design must include a reasonable probability that a structure will not collapse catastrophically under the effect of misuse or accident. Whilst it is impossible to allow for every eventuality, damage which is disproportionate to the original cause must be avoided. It may well be necessary to consider the possible occupancy or use of a structure in relation to the likely hazards to which the structure may be subject. Building Regulation A3[11] deals with disproportionate collapse and the Approved Document A[10] defines classes of building for which structural tying requirements, notional removal of structural elements and/or systematic risk analysis must be considered. The Approved Document recognises that BS 5628 Part 1[7] provides suitable guidance on tying and removal of structural elements in relation to unreinforced masonry structures. It is anticipated that in 2010 Approved Document A[10] will be updated to reflect the categorisation of building type and occupancy used in BS EN 1991-1-7[47], *Accidental Actions*. The Building Classes in Approved Document A[10] become Consequence classes in the BS EN[47]. Table B.1 is based upon the categorisation in BS EN 1991-1-7[47]. The minor differences from the equivalent table in Approved Document A[10] are indicated in notes **d** and **e** to Table B.1.

Table B.1 Categorisation of consequence classes

Consequence class	Example of categorisation of building type and occupancy
1	Single occupancy houses not exceeding 4 storeys. Agricultural buildings. Buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance of 1½ times the building height.
2a Lower Risk Group	5 storey single occupancy houses. Hotels not exceeding 4 storeys. Flats, apartments and other residential buildings not exceeding 4 storeys. Offices not exceeding 4 storeys. Industrial buildings not exceeding 3 storeys. Retailing premises not exceeding 3 storeys of less than 1000m^2 floor area in each storey[d]. Single storey educational buildings. All buildings not exceeding 2 storeys to which members of the public are admitted and which contain floor areas not exceeding 2000m^2 at each storey.

Table B.1 (Continued)	
2b Upper Risk Group	Hotels, flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys. Education buildings greater than single storey but not exceeding 15 storeys. Retailing premises greater than 3 storeys but not exceeding 15 storeys. Hospitals not exceeding 3 storeys. Offices greater than 4 storeys but not exceeding 15 storeys. All buildings to which members of the public are admitted which contain floor areas exceeding 2000m^2 but less than 5000m^2 at each storey. Car parking not exceeding 6 storeys.
3	All buildings defined above as Class 2 Lower and Upper Consequence class that exceed the limits on area and number of storeys. All buildings to which members of the public are admitted in significant numbers.[e] Stadia accommodating more than 5000 spectators. Buildings containing hazardous substances and/or processes.

Notes

a For buildings intended for more than one type of use the 'Consequence class' should be that relating to the most onerous type.

b In determining the number of storeys basement storeys may be excluded provided such basement storeys fulfil the requirements of 'Consequence class 2b Upper Risk Group'.

c This table is not exhaustive and can be adjusted.

d In Approved Document A[10], retail premises under Class 2A, have a limiting floor area of 2000m^2.

e The item 'All buildings to which members of the public are admitted in significant numbers' in Consequence class 3 above does not appear in Approved Document A[10].

B.2 Recommended strategies

Adoption of the following recommended strategies should provide a building with an acceptable level of robustness to sustain localised failure without a disproportionate level of collapse.

For buildings in Consequence class 1:

Provided a building has been designed and constructed in accordance with the rules given in EC6 Part 1-1[1] or BS 5628 Part 1[7] for satisfying stability in normal use, no further specific consideration is necessary with regard to accidental actions from unidentified causes.

For buildings in Consequence class 2a (Lower Group):

In addition to the recommended strategies for Consequence class 1, the provision of effective horizontal ties, or effective anchorage of suspended floors to walls, should be provided (see Table B.2).

For buildings in Consequence class 2b (Upper Group):

In addition to the recommended strategies for Consequence class 1, the provision of:

- horizontal ties, together with vertical ties (see Table B.3) in all supporting columns and walls should be provided, or alternatively,
- the building should be checked to ensure that upon the notional removal of each supporting column and each beam supporting a column, or any nominal section of load-bearing wall (one at a time in each storey of the building) the building remains stable and that any local damage does not exceed a certain limit. Approved Document A[10] requires that the area of floor at any storey at risk of collapse does not exceed 15% of the floor area of that storey or $70m^2$, whichever is smaller, and does not extend further than the immediate adjacent storeys.

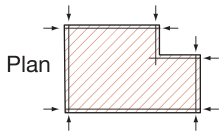
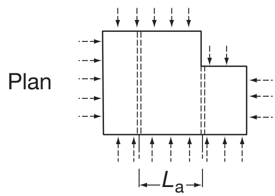
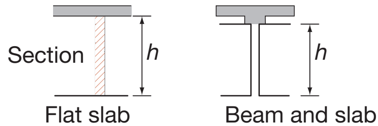
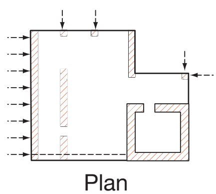
Where the notional removal of such columns and sections of walls would result in an extent of damage in excess of the agreed limit, or other such limit specified, then such limits should be designed as a 'key element'. A key element is a member that can withstand an accidental design load of $34kN/m^2$ without collapse. When considering the design of key elements or proving that structural elements may be removed without causing collapse, the partial factors used should be taken from BS EN 1991-1-7[47].

In the case of buildings of load-bearing wall construction, the notional removal of a section of wall, one at a time, is likely to be the most practical strategy to adopt.

For buildings in Consequence class 3:

A systematic risk assessment of the building should be undertaken taking into account both foreseeable and unforeseeable hazards.

Table B.2 Requirements for full peripheral, internal and column or wall ties

Type of tie	Unit of tie force	Size of design tie force[b]	Location of tie force (arrowed)
A. Peripheral	kN	F_t	Around whole perimeter 
B. Internal (both ways)	kN/m width	F_t or $\frac{F_t(G_k + Q_k)}{7.5} \frac{L_a}{5}$ F_t F_t or $\frac{F_t(G_k + Q_k)}{7.5} \frac{L_a}{5}$ whichever is the greater	**One way spans** (i.e. in cross wall or spine construction) i) in direction of span ii) in direction perpendicular to span **Two way spans (in both directions)**  
C. External column	kN	$2F_t$ or $(h/2.5)\ F_2$, whichever is the lesser, where h is in metres.	
D. External wall	kN/m length of load-bearing wall		

Notes

a This table is based on data from BS 5628 Part 1[7]

b Basic horizontal ties force = F_t = 60 kN or 20 + $4N_s$ kN, whichever is the lesser of the two values; N_s is the number of storeys (including ground and basement).

	Fixing requirements and notes
	• Ties should be: – placed within 1.2m of edge of floor or roof or in perimeter wall – anchored at re-entrant corners or changes of construction.
	• Internal ties should be anchored to perimeter ties. • Internal ties should be provided: – uniformly throughout floor or roof width; or, – concentrated (6m max. horizontal tie spacing); or, – within walls 0.5m max. above or below the floor or roof and at 6m max. horizontal spacing; – in addition to peripheral ties spaced evenly in perimeter zone. • Calculation of tie forces should assume: – $(G_k + Q_k)$ as the sum of average characteristic dead and imposed loads in kN/m^2 – L_a as the lesser of: – the greatest distance in metres in the direction of the tie, between the centres of columns or other vertical loadbearing members whether this distance is spanned by a single slab or by a system of beams and slabs; or – 5 x clear storey height h.
	• Corner columns should be tied in both directions. • Tie connections to masonry may be based on shear strength or friction (but not both). • Wall ties (where required) should be: – spaced uniformly along the length of the wall; – concentrated at centres not more than 5m apart and not more than 2.5m from the end of the wall. • External column and wall ties may be provided partly or wholly by the same reinforcement as perimeter and internal ties.

Table B.3 Requirements for full vertical ties	
Minimum thickness of a solid wall or one loadbearing leaf of a cavity wall	140mm
Minimum characteristic compressive strength of masonry	$5N/mm^2$
Maximum ratio h_a/t	20
Allowable mortar strength classes / designations (see *Manual* Table 3.3)	M4 (iii), M6 (ii), M12 (i)
Tie force	$\frac{34A}{8000}\left(\frac{h_a}{t}\right)^2$ N or 100kN/m length of wall or per column, whichever is the greater
Positioning of ties	5m centres max. along the wall and 2.5m max. from an unrestrained end of any wall

Notes

a A is the horizontal cross-sectional area in mm^2 of the column or wall including piers, but excluding the non-loadbearing leaf, if any, of an external wall of cavity construction.

b h_a is the clear height of a column or wall between restraining surfaces.

c t is the thickness of column or wall.

d This table is based on data from BS 5628 Part 1: 2005[7].

Manual for the design of plain masonry in building structures to Eurocode 6

This *Manual* supports the design of structures to BS EN 1996-1-1: 2005, BS EN 1996-1-2: 2005 and BS EN 1996-2: 2006 for construction in the UK. Nationally Determined Parameters from the UK National Annex have been taken into account in the design formulae that are presented.

The range of structures covered by the *Manual* is limited to building structures that do not rely on bending in masonry for their overall stability (e.g. sway frame buildings). However, the design of individual masonry elements subject to lateral loading and involving bending for their resistance is included. The structural design of reinforced and prestressed masonry is specifically excluded from the *Manual*, as are retaining walls and arched structures. The exception to this is the use of bed joint reinforcement in laterally loaded wall panels and for crack control. The design of both loadbearing masonry and masonry infill panels to framed structures in accordance with Eurocode 6 is covered by the *Manual*.

The *Manual* is similar in layout to the Institution's earlier manuals on British Standards and covers the following:

- choice of structural form (conceptual design)
- choice of materials
- general principles of limit state design for masonry walls and columns
- design of loadbearing masonry
- design of laterally loaded masonry
- details and construction
- design for fire.

This *Manual* is part of a suite of manuals for the Eurocodes.

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