The Institution of Structural Engineers

January 2005

# Manual for the design of plain masonry in building structures

**SECOND EDITION**

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**Manual for the design of plain masonry in building structures**
**(Second edition, 2005)**

Advisory Note, March 2011

The Institution brings to the attention of users of this Manual that being published in January 2005 it is based on, and refers to; **BS5628-1; 1992 and BS5628-3; 2001**.

Users should note that new editions of both BS5628-1 and BS5628-3 were published in December 2005 by the British Standards Institution. **The Institution recommends that this Manual should not be used in conjunction with information or data from the 2005 editions of these codes of practice except with great care and specific experience.**

The 2005 editions of BS5628-1 & BS5628-3 made changes to align these codes of practice with published supporting standards and the approach adopted in Eurocode 6, including reductions in the allowable shear strength and other adjustments relating to flexure. In addition clause numbering and headings were changed. It should also be noted that the Manual does not make provision for the 2004 revision of Approved Document A with respect to disproportionate collapse as this was not addressed in BS5628 until the edition published in December 2005.

**The Manual should not be used directly in conjunction with information, including characteristic compressive strengths, published in the 2003 revisions of BS EN 771, as this would result in a significant reduction in overall factors of safety.**

The 2003 editions of BS EN 771 were phased in gradually with the earlier British Standards being withdrawn in 2006. The user should note that this Manual refers specifically to the 1992 editions of parts of BS EN 771; the specifications for masonry units. The text of this Manual is correct when considering these 1992 editions of BS EN 771 and the 1992 and 2001 editions of BS5628.

Users of the Manual should note that it is based on the editions of the codes listed in the references section only, the Institution recommends that users should avoid using later editions of these codes of practice in conjunction with this Manual.

BS5628 was withdrawn in April 2010 along with all pre-existing British Standards and superseded by BS EN 1996. The Institution published *Manual for the design of plain masonry in building structures to Eurocode 6* in February 2008, which is based on and refers to BS EN 1996.

## Constitution

K C White FREng BSc(Eng) CEng FIStructE FICE FIHT *Chairman*
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A R Cusens OBE BSc(Eng) PhD DSc FREng CEng FIStructE FICE FRSE
R J W Milne† BSc *Secretary*

*deceased March 1989 ‡deceased July 2002 †deceased August 2002

The second edition was edited by N J Seward CEng FIStructE MICE
*Secretary* to the second edition B H G Cresswell Riol BEng

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Published by the Institution of Structural Engineers
11 Upper Belgrave Street, London SW1X 8BH, United Kingdom
Telephone: +44(0)20 7235 4535 Fax: +44(0)20 7235 4294
Email: mail@istructe.org.uk, Website: www.istructe.org.uk

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# Contents

# Foreword to the First Edition

The increasing use of masonry as a main structural medium for buildings alerted the Institution of Structural Engineers, some years ago, to the need for guidance on the design of plain (i.e. unreinforced) masonry structures.

The work was begun by a Task Group under the Chairmanship of Keith White FREng and culminated in a seminar held in October 1992 to discuss a draft document. This led to the formation of an Editorial Panel which prepared the *Manual* now being published.

The *Manual* provides design guidance which complies with BS5628 and can be applied normally to buildings up to four storeys high. It provides the basis for structural calculations for loadbearing masonry or for the masonry infill of a structural frame.

The work of the original Task Group is gratefully acknowledged. The members of the Editorial Panel carried out extensive redrafting of the *Manual* in response to comments made at the 1992 seminar and thereafter. I would like particularly to express gratitude for their contributions and also to thank all those whose knowledge and expressed views helped to shape this publication.

PROFESSOR A R CUSENS, OBE
Chairman, Editorial Panel

# Foreword to the Second Edition

Since the publication of the first edition in 1997, a number of amendments have been made to BS5268 parts 1 and 3, the underpinning British standards to this *Manual*. Revisions to this edition have therefore been made with reference to the current editions of these codes, BS5268-1, 2001 and BS5628-3, 2002.

In view of the gradual phased introduction of BS EN (CEN) product standards for masonry and the withdrawal of the conflicting and existing British standards, this revision also includes references to both sets of standards and covers the interim period where both sets of standards apply. References to BS EN Standards have been included where appropriate.

The opportunity has also been taken to update sections of the *Manual* in line with current practice, including masonry movement and mortar specification.

The overall scope of the *Manual* remains to provide a concise point of reference for the initial design of loadbearing masonry that complies with BS5268.

My thanks are expressed to fellow members of the Editorial Panel and the Institution secretariat for their hard work involved in this update.

Norman Seward.

NORMAN SEWARD C Eng FIStructE MICE
Editor, Second Edition

# 1 Introduction

## 1.1 Aims of the *Manual*

This *Manual* provides guidance on the design of plain masonry in building structures. Masonry designed in accordance with this *Manual* will normally comply with BS5628-1[1] and -3[2]. 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 reinforcement.

## 1.2 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 columns for their overall lateral stability. 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, retaining walls, freestanding walls and arched structures is specifically excluded from the *Manual*, as is the structural design for public buildings with clear spans of 9m or more.

While the *Manual* has been drafted for the design of new structures, the principles may be applicable to alterations to existing structures. However, care should be taken when assessing the characteristic strength of existing masonry.

For structures or elements outside this scope BS5628 should be used.

## 1.3 Use of the *Manual*

The *Manual* is intended to be used by structural engineers in the preparation of structural design calculations. The first decision to be made is whether to adopt a loadbearing masonry design or to provide a structural frame with masonry infill. Depending on the decision made, one of the routes shown in Fig. 1 should be followed.

Changes from the first edition to the second edition are noted with a line in the margin.

## 1.4 Contents of the *Manual*

The *Manual* covers the following design stages:

- choice of structural form
- choice of materials
- general principles of limit-state design for masonry walls and columns
- design of loadbearing masonry
- details and construction.

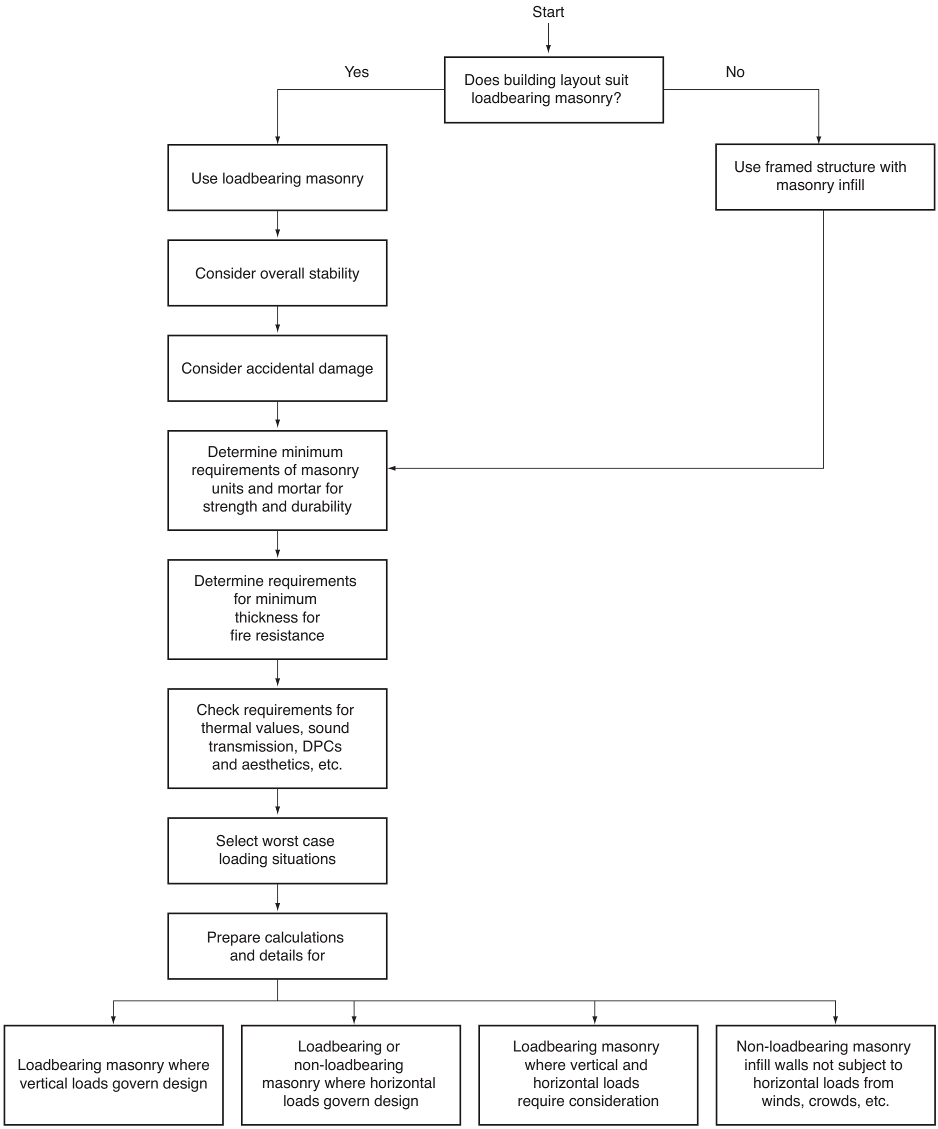



**Fig. 1 Alternative routes for masonry design**

# 2 Choice of structural form

## 2.1 General

One engineer should be responsible for the overall design, including stability, and consider the compatibility of the design and detailing of parts and components, even where some or all of the design and details of those parts and components are not made by the same engineer.

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 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 minimal 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 the standard sizes of masonry units and courses 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 depend on 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 (Fig. 2). Where this is not possible, the resultant additional twisting moments must be considered when calculating the load carried by each strongpoint (Fig. 3).

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.

## 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 (Fig. 4). 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 strongpoints and crosswalls relies on the floors acting as horizontal diaphragms (Fig. 5).

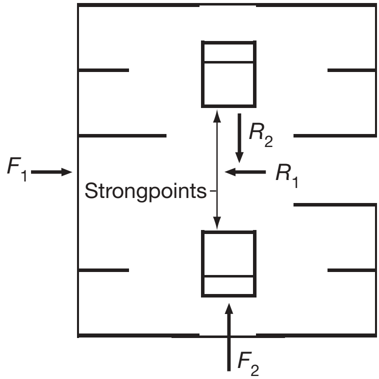



**Fig. 2 Symmetrical plan strongpoints**

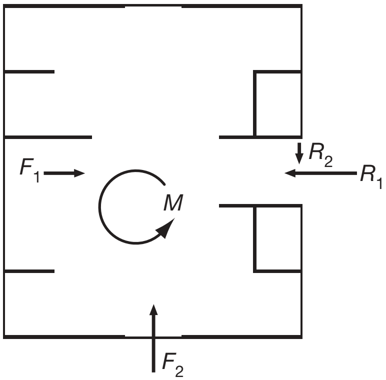



**Fig. 3 Asymmetrical plan strongpoints**

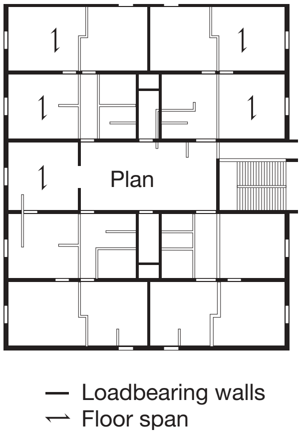



**Fig. 4 Cellular wall plan**

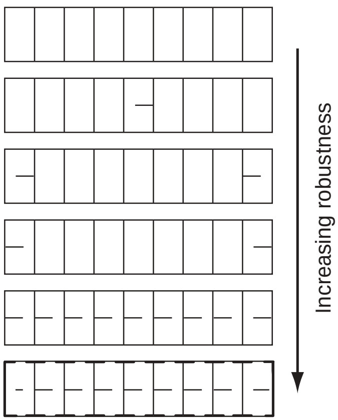



**Fig. 5 Crosswall plan**

### 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 gable walls. The combined strength of strongpoints and spine-walls relies on the floors acting as horizontal diaphragms (Fig. 6).

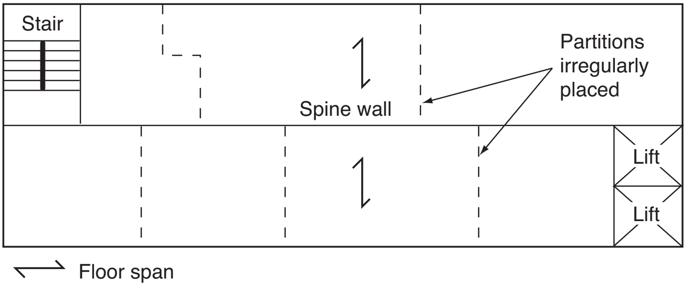



**Fig. 6 Spine-wall plan**

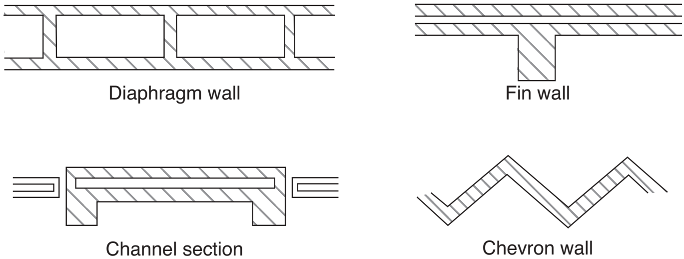



**Fig. 7 Typical geometric wall sections**

## 2.6 Geometric sections

Geometric profiles can be readily formed in masonry and are particularly suitable for use in tall and slender single-storey 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 flexural stiffness in the plan shape of the wall elements (Fig. 7). The section should be sized to suit brick and block unit dimensions to avoid excessive cutting and poor bonding.

For basic design guidance on such walls refer to subsection 5.4.6.

## 2.7 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 identified or the element in question strengthened. 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).

Generally the cellular plan form produces the most stable and robust structure. Nevertheless, 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 and is basically the least robust type of structure. 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 (Fig. 6).

In the case of both spine-wall and crosswall construction the location of building primary construction joints and also of secondary masonry elemental movement joints is of significance 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.

In all forms of movement joint it is essential to provide a continuous joint through any finishes (e.g. plaster), attached cladding and similar elements.

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 (Fig. 8) and be in one plane. Consideration should be given to the need to carry the joint through the foundations.

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 brickwork generally exhibits long-term moisture expansion, whereas concrete blocks and bricks and calcium silicate bricks experience drying shrinkage. In long walls, concrete and calcium silicate masonry may require the inclusion of expansion joints.

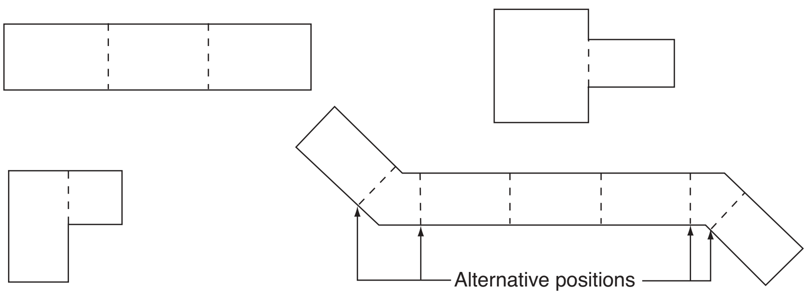



**Fig. 8 Location of primary movement joints**

The spacing and location of secondary elemental movement joints need to be carefully considered. Features of the building that should be considered for 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.

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. The outer leaf should, therefore, be supported at intervals of not more than every third storey or 9m, whichever is less. However for buildings not exceeding 4 storeys or 12m in height, whichever is less, the outer leaf may be uninterrupted for its full height (see clause 29.2.2 of BS5628-1[1]). Alternatively, calculations may be carried out (see clause 29.2.3 of BS5628-1), and the effects of horizontal movement should also be considered (see clause 29.2.4 of BS5628-1). Further guidance may be obtained from CIRIA Technical Note 107[3], BDA Design Note 10[4], BS5628-3[2] Annex B. Information may also be found in BRE Digests 227[5], 228[6] and 229[7].

### 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 support deflections
- 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 units.

Particularly in cases of precast concrete floor units and timber floor joists and roof trusses (Fig. 9) the designer must be satisfied that the 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.

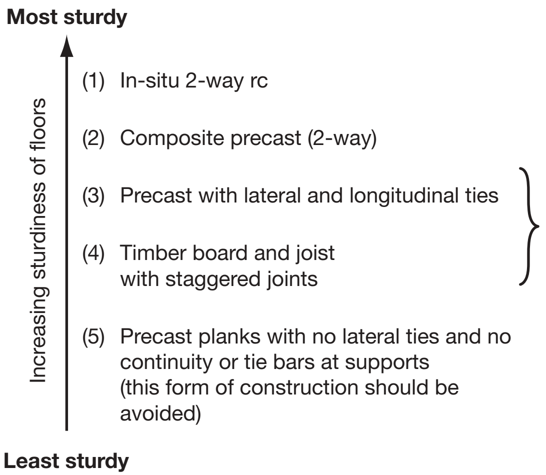



**Fig. 9 Sturdiness of floors**

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.

## 2.10 Infill masonry to framed structures

Masonry infilling may be used to provide the bracing to reinforced concrete or steel framed structures. This method is not however in widespread current use. In such circumstances 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 the effects of possible removal of such 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 brick or concrete brick or block masonry panel 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.

Current practice is to use infill masonry panels that 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 adjacent walls (above and below) is not impaired. 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.

Openings can have a major influence on the load paths in masonry walls, affecting the stress distribution.

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 (Fig. 10).

The local effect of an opening is to cause increased stresses under lintel bearings etc. (Fig. 11). Reference should be made to BS5977-1[8] to assess the load to be carried by the lintel.

Care should be exercised where openings are formed in walls that provide support or a restraining effect to an adjacent wall. A minimum uninterrupted construction length of ten times the thickness of the supported wall, measured from the internal interface of the wall junction, should be provided in the restraining wall (see clause 28.2 of BS5628-1[1] and clause 5.2.3.2.2 of BS5268-3[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.

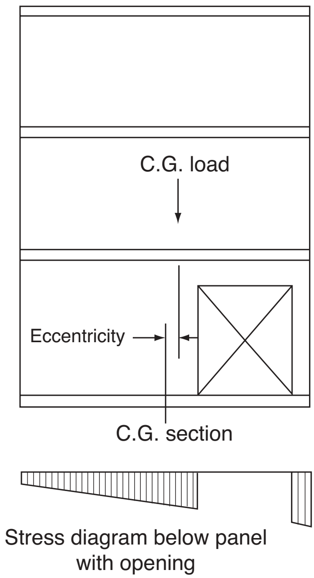



**Fig. 10 Stress arising from eccentric loading caused by the position of the opening**

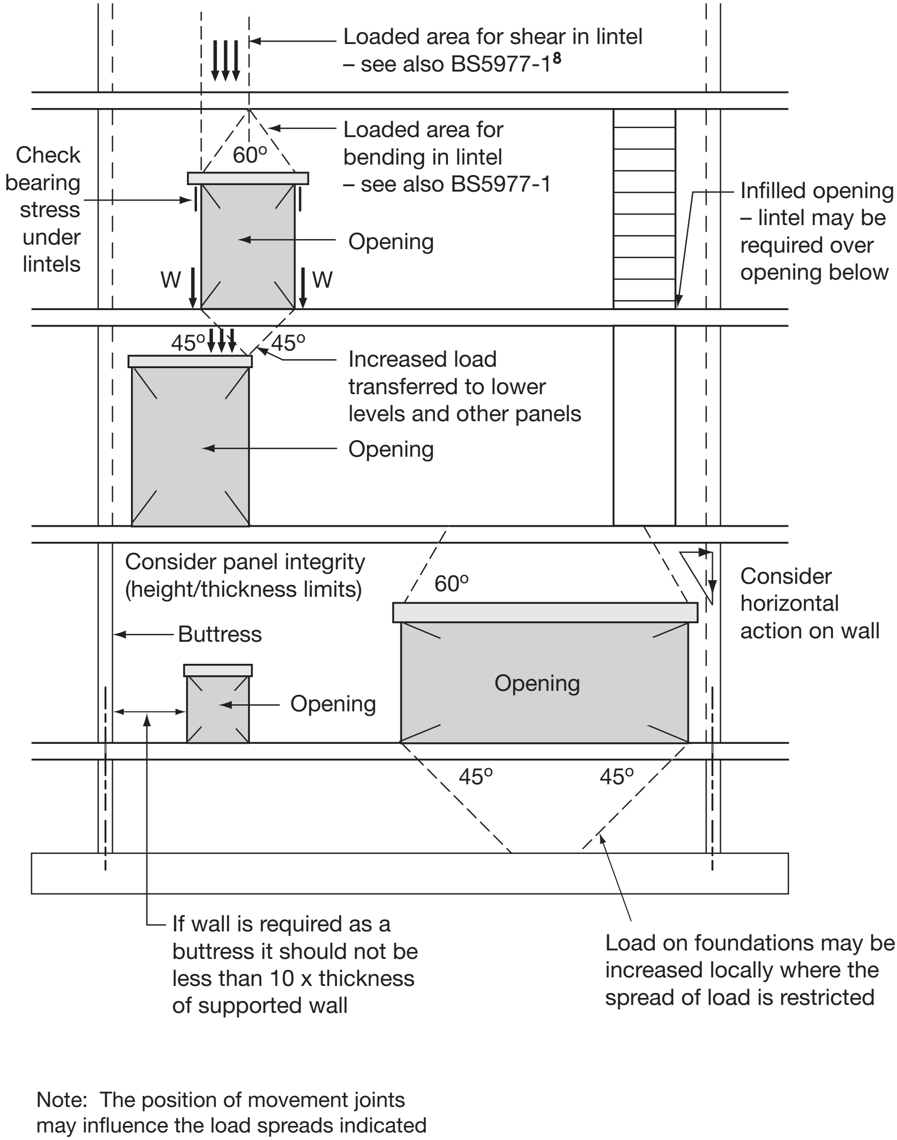



**Fig. 11 Load paths arising from lintels, openings, etc.**

# 3 Choice of materials

## 3.1 Design co-ordination

The engineer should be satisfied that the materials 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 bricks.

Neither BS5628 nor the individual British Standards relevant to materials concern themselves with robustness, i.e. resistance to impact and abrasion, or the provisions of secure fixings for the attachment of other components. For example, hollow or lightweight blockwork is less resistant to impact or abrasion, which could be a consideration in certain situations such as factories and warehouses.

## 3.2 Structural masonry units

Before a final choice of masonry unit is made, it may be prudent to obtain test results of recent production runs, and evidence of suitability in the form of completed buildings using the units. Where no recent test certificates are available, tests may be carried out to demonstrate that the units satisfy the engineering requirements.

Structural masonry units should comply with the relevant British Standards, where applicable the replacement BS EN standards are shown in parenthesis:

- Calcium silicate (sandlime and flintlime) bricks BS1879[9] (BS EN 771-2[10])
- Clay bricks BS3921[11] (BS EN 771-1[12])
- Dimensions of bricks of special shapes and sizes BS 4729[13]
- Natural Stone masonry BS5628-3[2] (BS EN 771-6[14])
- Precast concrete masonry units BS6073-1[15] (BS EN 771-3[16] and BS EN 771-4[17])
- Reconstructed stone masonry units BS6457[18] (BS EN 771 - 5[19])
- Clay and calcium silicate modular bricks BS6649[20]

The supply of units should be monitored to ensure compliance with the specification is being achieved. A guide to the properties of masonry materials is given in Table 1.

## 3.3 Mortars and mortar joints

### 3.3.1 Mortars

Mortars should be selected on the grounds of strength, durability, compatibility and economy. Where cracking is likely to occur, the use of strong (cement-rich) mortars with weak units can give rise to cracking of the units and should generally be avoided (see Table 2). In cavity walls the choice of the same mortar for each leaf is preferred.

Designers should be aware of the possible need for cubes or prisms to be taken to monitor the strength and consistency of mortars, guidance for which may be obtained from BS5628-1[1].

Certain fine sands, while conforming to BS EN 13139[21], may require further adjustment of mix proportions (see BS5628-3[2], Table A1). Choice and grading of the sand has a significant effect on workability and on the final properties of the hardened mortar.

'Masonry cement' is a mixture of Portland cement, inert fillers and may contain additives. Because of its likely misuse its adoption in structural masonry is not advised.

| **Table 1 Guide to the properties of masonry** | | | | | | |
|---|---|---|---|---|---|---|
| Properties | Clay brickwork | Calcium silicate brickwork | Dense concrete blockwork | Light-weight concrete blockwork | Aerated concrete blockwork | Natural limestone |
| Weight (kN/m³) | 16 – 22 | 20 | 15 – 21 | 7 – 16 | 4 – 9 | 22 |
| Compressive strength (N/mm²) | 5 – 104 | 14 – 48.5 | 7 – 40 | 3.5 – 10.5 | 2.8 – 10 | 10 – 50 |
| Flexural strength (N/mm²) | | | | | | |
| – plane of failure perpendicular to bed joints | 1.5 | 0.9 | 0.6 | 0.6 | 0.6 | – |
| – plane of failure parallel to bed joints | 0.5 | 0.3 | 0.25 | 0.25 | 0.25 | – |
| Elastic modulus (N/mm²) | 5 – 25 or 450-900 $f_k$[a] | 14 – 18 | 10 – 25 or 300 $f_k$[a] | 4 – 16 | 1.7 – 8 | 15 |
| Creep factor – final creep strain to elastic strain at working stresses | 1.2 – 4.0 | – | 2.0 – 7.0 | | 2.0 | – |
| Reversible moisture movement (%) | 0.02 (+) | 0.01 – 0.05 (+) | 0.02 – 0.06 (–) | 0.03 – 0.06 (–) | 0.02 – 0.03 (–) | 0.01 (+) |
| Initial moisture expansion (+) or drying shrinkage (–) (%) | 0.02 – 0.10 (+) (0.03 when on site) | 0.01 – 0.05 (–) | 0.02 – 0.06 (–) | 0.05 – 0.06 (–) | 0.05 – 0.09 (–) | 0.01 |
| Coefficient of thermal expansion (x10⁻⁶/ºC) | 5 – 8 | 8 – 14 | 6 – 14 | 7 – 12 | 8 | 4 |
| Long-term natural water absorption (%) | 3.0 – 15.0 | | | | | |
| Thermal conductivity at 5% moisture content (W/mºC) | 0.07 – 1.20 | 1.2 | 0.6 – 1.3 | 0.20 – 0.44 | 0.10 – 0.27 | 1.3 |

Note
**a** Broadly but not linearly related to $f_k$, the characteristic compressive strength.

Plasticisers are often used in lieu of lime to improve the workability and durability of mortars. They do not, however, provide the extra gain of strength with time possible with some limes. Plasticisers should be used only in accordance with the manufacturer's instructions, with particular consideration given to the height of lifts and the risk of loosened ties. When used in conjunction with retarders, care should be taken to avoid incompatibility.

Care should 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 retarded, ready-to-use mortar. This is weight-batched at the factory and delivered to site containing a set retarder and usually air entrainer to enable the mortar to be used over a maximum 48-hours working period. On site, silo-supplied mortar facilities are also becoming popular, as they allow freedom of mortar provision to the point of use. 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.

**Table 2 Mortar mixes**

<table>
<tr><td colspan="2" rowspan="2">See BS5628-1[1]<br>Table 1</td><td rowspan="2">Mortar designation</td><td colspan="2">Types of mortar (proportion by volume)</td><td colspan="2">Mean compressive strength at 28 days ($N/mm^2$)</td></tr>
<tr><td>Cement: lime: sand</td><td>Cement: sand with plasticiser</td><td>Preliminary (laboratory) tests</td><td>Site tests</td></tr>
<tr><td>Increasing strength ↑</td><td>Increasing ability to accommodate movement ↓</td><td>(i)<br>(ii)<br>(iii)<br>(iv)</td><td>1: 0 to ¼: 3<br>1: ½: 4 to 4½<br>1: 1: 5 to 6<br>1: 2 : 8 to 9</td><td>–<br>1: 3 to 4<br>1: 5 to 6<br>1: 7 to 8</td><td>16.0<br>6.5<br>3.6<br>1.5</td><td>11.0<br>4.5<br>2.5<br>1.0</td></tr>
<tr><td colspan="3">Direction of change in properties is shown by the arrows</td><td colspan="2">Increasing resistance to frost attack during construction →<br>Improvement in bond and consequent resistance to rain penetration ←</td><td colspan="2"></td></tr>
</table>

Notes (see BS5628-3[2] Table 15)

1 Where mortars are to be specified by strength or where special category construction is to be used, the proportions should be determined from tests (see Appendix A of BS5628-1).

2 Mixes of the same designation have approximately equivalent strength and durability.

3 The different types of mortar that comprise any one designation are approximately equivalent in compressive strength and do not generally differ greatly in their own properties. Some general differences between types of mortar are indicated by the arrows at the bottom of the table, but these differences can be reduced (see BS5628-3: clause 5.7.1).

4 The range of sand contents is to allow for the effects of the differences in grading upon the properties of the mortar. In general, the values apply to Type G of BS EN 13139[21] whilst the higher values apply to Type S. Mortars incorporating both lime and air-entrainment can be used with any sands within the BS EN 13139 gradings.

5 Air entrainment to improve the durability and the working properties of the mortar is recommended. It may be achieved by the use of air-entrained cements or improved cements, by the addition of plasticiser to the site mixer, or by the use of a factory made mortar.

### 3.3.2 Mortar joints

Mortar joints may be finished in a number of ways. When this 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, 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.

For all types of masonry, it is essential 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.

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

The principal types of joint profile used for brick and block masonry are shown in Fig. 12.

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 brick 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 brickwork. 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 bricks and thus to penetrate the bricks and joints causing discolouration and possible front damage. For these reasons it should be confined to lightly stressed interior walls and external brickwork using appropriate quality bricks.

**Square recessed**
This joint, when used with durable bricks, 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 bricks 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 bricks are unsuitable for overhung struck and square recessed jointing.

**Fig. 12 Principal types of joint profile for brick and block masonry**

## 3.4 Durability

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.

### 3.4.1 Frost resistance

Water saturation is the 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.

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 by copings, sills, and roofs with adequate overhangs and drips. The provision of dpcs 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.

Manufacturers of clay bricks are required to state the frost resistance of their products by classifying them from experience in use as:

- F: which means frost resistant, even when used in exposed positions where they will be liable to freezing while saturated. If there is any doubt it is strongly recommended that the manufacturer's advice as to the suitability of the product should be sought
- M: which means moderately frost resistant and suitable for general use in walling that is protected from saturation
- O: which means not frost resistant. Such bricks are seldom made deliberately; they may be adequate for internal walls.

### 3.4.2 Soluble salt content

Most clays used in brickmaking contain soluble salts that may have been retained in the fired bricks. If brickwork becomes saturated for long periods, soluble sulphates will be released. These may cause mortars that have been incorrectly specified or batched and have a low Portland cement content to deteriorate under sulphate attack. Sulphates from the ground or other sources may be equally destructive.

Some clay bricks meet limits placed on the level of certain soluble salts and these are designated 'L', signifying low soluble salt content. Those that do not meet the limits are designated as 'N' for normal soluble salt content. The use of designation 'L' rather than 'N' bricks in clay brickwork that may remain saturated for long periods should reduce the risk of sulphate attack in the mortar. Sulphate attack can also occur on concrete masonry units and joints in the presence of external sulphates, e.g. dissolved in groundwater.

The most effective way of reducing the risk of sulphate attack is by designing to prevent saturation and where this is unavoidable to use the higher designation (i) and (ii) mortars, i.e. those rich in Portland cement. The use of mortars rich in Portland cement should, however, be balanced against the increased risk of cracking (see subsection 3.3.1). In some cases the use of sulphate-resisting Portland cement is advised.

### 3.4.3 Efflorescence

Efflorescence is a crystalline deposit left on the surface of clay and concrete masonry after the evaporation of water carrying dissolved soluble salts. Manufacturers have to state which category the bricks being offered correspond to when subject to the efflorescence test described in BS3921[11], namely nil, slight or moderate. Specifiers should be aware that these categories do not relate to the degree of efflorescence to which brickwork may be liable in certain site conditions. The risk of efflorescence, which is harmless and, whilst unsightly, usually temporary, can be best minimised by protecting the bricks in the stacks as well as newly built brickwork from rain (see BS5628-3[2] section A.4.1.3.2).

The minimum qualities of units and mortar that will provide adequate durability are tabulated in BS5628-3, extracts from which are reproduced as Table 3. Further information on brickwork durability can be obtained from BDA Design note 7[22].

**Table 3 Durability of masonry in finished construction (extract from BS5628-3: Table 13)**

| (A) Work below or near external ground level | | | | | |
|---|---|---|---|---|---|
| Masonry condition or situation | Quality of masonry units and appropriate mortar designations | | | | Remarks (references in this table are to BS5628-3[2]). |
| | Clay units | Calcium silicate units | Concrete bricks | Concrete blocks | |
| A1 Low risk of saturation with or without freezing | FL, FN, ML, or MN in (i), (ii) or (iii) | Classes 3 to 7 in (iii) or (iv) (see remarks) | $\geqslant$ 15 N/mm$^2$ in (iii) | (a) of net density $\geqslant$ 1500kg/m$^3$; or<br>(b) made with dense aggregate conforming to BS882[23] or BS1047[24]; or<br>(c) having a compressive strength $\geqslant$ 7 N/mm$^2$; or<br>(d) most types of autoclaved aerated block (see remarks) in (iii) | Some types of autoclaved aerated concrete block may not be suitable. The manufacturer should be consulted.<br>If sulphate ground conditions exist, the recommendations in clause 5.6.4 should be followed.<br>Where designation (iv) mortar is used it is essential to ensure that all masonry units, mortar and masonry under construction are protected fully from saturation and freezing (see clause A.4.1.3.2 and clause A.5.1.1).<br>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 may remain wet for long periods of time, particularly in winter. Where FN or MN clay units are used in A2 or A3, sulphate-resisting cement should be used (see clause 5.6.4).<br>In conditions of highly mobile groundwater, consult the manufacturer on the selection of materials (see clause 5.6.1.4). |
| A2 High risk of saturation without freezing | FL, FN, ML, or MN in (i) or (ii) (see remarks) | Classes 3 to 7 in (ii) or (iii) | $\geqslant$ 15N/mm$^2$ in (ii) or (iii) | As for A1 in (ii) or (iii) | |
| A3 High risk of saturation with freezing | FL or FN in (i) or (ii) | Classes 3 to 7 in (ii) | $\geqslant$ 20N/mm$^2$ in (ii) or (iii) | As for A1 in (ii) | |

**Table 3 (Continued)**

| Masonry condition or situation | Quality of masonry units and appropriate mortar designations: Clay units | Calcium silicate units | Concrete bricks | Concrete blocks | Remarks (references in this table are to BS5628-3[2]). |
|---|---|---|---|---|---|
| **(B) DPCs** | | | | | |
| B1 In building | DPC 1 as described in BS 3921, in (i) | Not suitable | Not suitable | Not suitable | Masonry DPCs can resist rising damp but will not resist water percolating downwards. If sulfate ground conditions exist, the recommendations of clause 5.6.4 should be followed. DPCs of clay units are unlikely to be suitable for walls of other masonry units, as differential movement can occur (see clause 5.4). |
| B2 In external works | DPC 2 as described in BS 3921, in (i) | Not suitable | Not suitable | Not suitable | |
| **(C) Unrendered external walls (other than chimneys, cappings, copings, parapets or sills)** | | | | | |
| C1 Low risk of saturation | FL, FN, ML or MN in (i), (ii) or (iii) | Classes 2 to 7 in (iii) or (iv) (see remarks) | $\geqslant 7N/mm^2$ in (iii) | 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 (see clause 5.5.4). Certain architectural features, e.g. brick masonry below large glazed areas with flush sills, increase the risk of saturation (see clause 5.6.5).<br>Where designation (iv) mortar is used it is essential to ensure that all masonry units, mortar and masonry under construction are protected fully from saturation and freezing (see clause A.4.1.3.2 and clause A.5.1.1).<br>Where FN clay units are used in designation (ii) mortar for C2, sulphate-resisting cement should be used (see clause 5.6.4). |
| C2 High risk of saturation | FL or FN in (i) or (ii) (see remarks) | Classes 2 to 7 in (iii) | $\geqslant 15N/mm^2$ in (iii) | Any in (iii) | |

**Table 3 (Continued)**

| (D) Rendered external walls (other than chimneys, cappings, copings, parapets or sills) | | | | | |
|---|---|---|---|---|---|
| Masonry condition or situation | Quality of masonry units and appropriate mortar designations | | | | Remarks (references in this table are to BS5628-3[2]). |
| | Clay units | Calcium silicate units | Concrete bricks | Concrete blocks | |
| Rendered external walls (other than chimneys, cappings, copings, parapets or sills) | FN or MN in (i) or (ii) (see remarks) or FL or ML in (i), (ii) or (iii) | Classes 2 to 7 in (iii) or (iv) (see remarks) | $\geqslant$ 7N/mm$^2$ in (iii) | Any in (iii) or (iv) (see remarks) | Rendered walls are usually suitable for most wind driven rain conditions (see clause 5.5.4). Where FN or MN clay units are used, sulphate-resisting cement should be used in the mortar and in the base coat of the render (see clause 5.6.4)<br>Where designation (iv) mortar is used it is essential to ensure that all masonry units, mortar and masonry under construction are protected fully from saturation and freezing (see clause A.4.1.3.2 and clause A.5.1.1). |
| (E) Internal walls and inner leaves of cavity walls | | | | | |
| Internal walls and inner leaves of cavity walls | FL, FN, ML, MN, OL or ON in (i), (ii), (iii) or (iv) (see remarks) | Classes 2 to 7 in (iii) or (iv) (see remarks) | $\geqslant$ 7N/mm$^2$ in (iv) (see remarks) | Any in (iii) or (iv) (see remarks) | Where designation (iv) mortar is used it is essential to ensure that all masonry units, mortar and masonry under construction are protected fully from saturation and freezing (see clause A.4.1.3.2 and clause A.5.1.1.). |

**Table 3 (Continued)**

(F) Unrendered parapets (other than cappings and copings)

| Masonry condition or situation | Quality of masonry units and appropriate mortar designations | | | | Remarks (references in this table are to BS5628-3[2]) |
|---|---|---|---|---|---|
| | Clay units | Calcium silicate units | Concrete bricks | Concrete blocks | |
| F1 Low risk of saturation, e.g. low parapets on some single storey buildings | FL, FN, ML or MN in (i), (ii) or (iii) | Classes 3 to 7 in (iii) | $\geqslant$ 20N/mm$^2$ in (iii) | (a) of net density $\geqslant$ 500kg/m$^3$;<br>or<br>(b) made with dense aggregate conforming to BS882[23] or BS1047[24];<br>or<br>(c) having a compressive strength $\geqslant$ 7N/mm$^2$;<br>or<br>(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.<br>Some types of autoclaved aerated concrete block may not be suitable. The manufacturer should be consulted.<br>Where FN clay units are used in F2, sulphate-resisting cement should be used (see clause 5.6.4). |
| F2 High risk of saturation, e.g. where a capping only is provided for the masonry | FL or FN in (i) or (ii) | Classes 3 to 7 in (iii) | $\geqslant$ 20N/mm$^2$ in (iii) | As for F1 in (ii) | |

**Table 3 (Continued)**

| Masonry condition or situation | Quality of masonry units and appropriate mortar designations | | | | Remarks (references in this table are to BS5628-3[2]) |
|---|---|---|---|---|---|
| | Clay units | Calcium silicate units | Concrete bricks | Concrete blocks | |
| **(G) Rendered parapets (other than cappings and copings)** | | | | | |
| Rendered parapets (other than cappings and copings) | FN or MN in (i) or (ii) (see remarks) or FL or ML in (i), (ii) or (iii) | Classes 3 to 7 in (iii) | $\geqslant$ 7N/mm$^2$ in (iii) | Any in (iii) | Single-leaf walls should be rendered only on one face, the other being left free to breathe. All parapets should be provided with a coping. Where FN or MN clay units are used, sulphate-resisting cement should be used in the mortar and in the base coat of the render (see clause 5.6.4). |
| **(H) Chimneys** | | | | | |
| H1 Unrendered with low risk of saturation | FL, FN, ML or MN in (i), (ii) or (iii) | Classes 3 to 7 in (iii) | $\geqslant$ 10N/mm$^2$ in (iii) | Any in (iii) | Chimney stacks are normally the most exposed masonry on any building. Due to the possibility of sulfate attack from flue gases the use of sulfate-resisting cement in the mortar and in any render is strongly recommended (see clause 5.6.4). Brick masonry and tile capping 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. |
| H2 Unrendered with high risk of saturation | FL or FN in (i) or (ii) | Classes 3 to 7 in (iii) | $\geqslant$ 15N/mm$^2$ in (iii) | (a) of net density $\geqslant$ 1500kg/m$^3$; or<br>(b) made with dense aggregate conforming to BS882[23] or BS1047[24]; or<br>(c) having a compressive strength $\geqslant$ 7N/mm$^2$; or<br>(d) most types of autoclaved aerated block (see remarks) in (ii) | |
| H3 Rendered | FL or ML in (i), (ii) or (iii) or FN or MN in (i) or (ii) | Classes 3 to 7 in (iii) | $\geqslant$ 15N/mm$^2$ in (iii) | Any in (iii) | |

**Table 3 (Continued)**

(I) Cappings, copings and sills

| Masonry condition or situation | Quality of masonry units and appropriate mortar designations | | | | Remarks (references in this table are to BS5628-3[2]) |
|---|---|---|---|---|---|
| | Clay units | Calcium silicate units | Concrete bricks | Concrete blocks | |
| Cappings, copings and sills | FL or FN in (i) | Classes 4 to 7 in (ii) | $\geqslant$ 30N/mm$^2$ in (ii) | (a) of net density $\geqslant$ 1500kg/m$^3$;<br>or<br>(b) made with dense aggregate conforming to BS882[23] or BS1047[24];<br>or<br>(c) having a compressive strength $\geqslant$ 7N/mm$^2$;<br>or<br>(d) most types of autoclaved aerated block (see remarks) in (ii) | Some autoclaved aerated concrete blocks may be unsuitable for use in I. The manufacturer should be consulted.<br>Where cappings or copings are used for chimney terminals, the use of sulphate-resisting cement is strongly recommended (see clause 5.6.4).<br>Dpcs for cappings, copings and sills should be bedded in the same mortar as the masonry units. |

**Table 3 (Continued)**

| (J) Freestanding boundary and screen walls (other than cappings and copings) | | | | | |
|---|---|---|---|---|---|
| Masonry condition or situation | Quality of masonry units and appropriate mortar designations | | | | Remarks (references in this table are to BS5628-3[2]) |
| | Clay units | Calcium silicate units | Concrete bricks | Concrete blocks | |
| J1 With coping | FL or MN in (i) or (ii); or FL or ML in (i), (ii) or (iii) | Classes 3 to 7 in (iii) | $\geqslant$15N/mm$^2$ in (iii) | Any in (iii) | Masonry in free-standing walls is likely to be severely exposed, irrespective of climatic conditions. Such walls should be protected by a coping wherever possible and dpcs should be provided under the copings and at the base of the wall (see clause 5.5).<br>Where FN or MN clay units are used for J1 in conditions of severe driving rain (see clause 5.5), the use of sulphate-resisting cement is strongly recommended (see clause 5.6.4).<br>Where designation (iii) mortar is used for J2, the use of sulphate-resisting cement is strongly recommended (see clause 5.6.4).<br>Some types of autoclaved aerated concrete block may also be unsuitable. The manufacturer should be consulted. |
| J2 With capping | FL or FN (i) or (ii) (see remarks) | Classes 3 to 7 in (iii) | $\geqslant$ 20N/mm$^2$ in (iii) | (a) of net density $\geqslant$1500kg/m$^3$; or<br>(b) made with dense aggregate conforming to BS882[23] or BS1047[24]; or<br>(c) having a compressive strength $\geqslant$7N/mm$^2$ (see remarks); or<br>(d) most types of autoclaved aerated block (see remarks) in (iii). | |

**Table 3 (Continued)**

(K) Earth-retaining walls (other than cappings and copings)

| Masonry condition or situation | Quality of masonry units and appropriate mortar designations | | | | Remarks (references in this table are to BS5628-3[2]) |
|---|---|---|---|---|---|
| | Clay units | Calcium silicate units | Concrete bricks | Concrete blocks | |
| K1 With waterproofing on retaining face and coping | FL, FN, ML or MN in (i) or (ii) | Classes 3 to 7 in (ii) or (iii) | $\geqslant$15N/mm$^2$ in (ii) | (a) of net density $\geqslant$ 1500kg/m$^3$; or<br>(b) made with dense aggregate conforming to BS882[23] or BS1047[24]; or<br>(c) having a compressive strength $\geqslant$ 7N/mm$^2$; or<br>(d) most types of autoclaved aerated block (see remarks) in (iii) | Because of possible contamination from the ground and saturation by ground waters, in addition to subjection to severe climatic exposure, masonry in retaining walls is particularly prone to frost and sulphate attack. Careful choice of materials in relation to the methods for exclusion of water recommended in clause 5.5 is essential.<br>It is strongly recommended that such walls be backfilled with free-draining materials. The provision of an effective coping with a dpc (see clause 5.5) and waterproofing of the retaining face of the wall (see clause 5.6.1.4) is desirable.<br>Where FN or MN clay masonry units are used, the use of sulphate-resisting cement may be necessary (see clause 5.6.4).<br>Some types of autoclaved aerated concrete block are not suitable for use in K1. The manufacturer should be consulted.<br>Some concrete blocks are not suitable for use in K2. The manufacturer should be consulted. |
| K2 With coping or capping but no waterproofing on retaining face | FL or FN in (i) | Classes 4 to 7 in (ii) | $\geqslant$30N/mm$^2$ in (i) or (ii) | As for K1 but in (i) or (ii) (see remarks) | |

**Table 3 (Continued)**

(L) Drainage and sewerage, e.g. inspection chambers, manholes

| Masonry condition or situation | Quality of masonry units and appropriate mortar designations | | | | Remarks (references in this table are to BS5628-3[2]) |
|---|---|---|---|---|---|
| | Clay units | Calcium silicate units | Concrete bricks | Concrete blocks | |
| L1 Surface water | Engineering bricks, FL, FN, ML or MN in (i) (see remarks) | Classes 3 to 7 in (ii) and (iii) | ⩾20N/mm² in (iii) | (a) of net density ⩾ 1500kg/m³; or<br>(b) made with dense aggregate conforming to BS882[23] or BS1047[24] or<br>(c) having a compressive strength ⩾ 7N/mm²; or<br>(d) most types of autoclaved aerated block (see remarks) in (iii) | Where FN or MN clay masonry units are used, sulphate-resisting cement should be used.<br>If sulphate ground conditions exist the recommendations in clause 5.6.4 should be followed.<br>Some types of autoclaved aerated concrete block are not suitable for use in L1. The manufacturer should be consulted.<br>Some types of calcium silicate brick are not suitable for use in L2 or L3. The manufacturer should be consulted. |
| L2 Foul drainage (continuous contact with masonry) | Engineering bricks, FL, FN, ML or MN in (i) | Class 7 in (ii) (see remarks) | ⩾ 40N/mm² with cement content ⩾ 350kg/m³ in (i) or (ii) | Not suitable | |
| L3 Foul drainage (occasional contact with masonry) | Engineering bricks, FL, FN, ML or MN in (i) | Classes 3 to 7 in (ii) and (iii) (see remarks) | ⩾ 40N/mm² with cement content ⩾ 350kg/m³ in (i) or (ii) | Not suitable | |

### 3.4.4 Concrete blocks/bricks

**Table 4 General guidance on the recommended qualities of blocks, bricks and mortar to ensure durability**

| Location | Minimum strength of concrete unit ($N/mm^2$) | | Mortar designation |
|---|---|---|---|
| | Blocks | Bricks | |
| General internal; external above dpc | any block | 15 | (iii) or (iv) |
| External below dpc; free-standing walls; parapets | 3.5 dense; 7.0 lightweight | 20 | (iii) |
| Earth-retaining walls | 7.0 dense | 30 | (ii) |
| Sills and copings | consult manufacturer | 30 | (ii) |

Note
Where sulphate attack can occur, the use of sulphate-resisting cement may be necessary.

### 3.4.5 Lime bloom

Lime bloom is a white stain occurring on concrete 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. Brickwork built in wet weather is sometimes susceptible to this form of staining.

## 3.5 Fire resistance

If the required fire resistance of a loadbearing cavity wall with a thickness taken from Table 5 is more than 2 hours, the imposed load should be shared by both leaves; otherwise, if the load is carried only by the leaf exposed to the fire, the minimum thickness of that leaf should be that given for loadbearing single-leaf walls.

In order for a structural member to be able to carry its load during and after a fire its thickness may need to be greater than that which is dictated by purely structural considerations.

## 3.6 Wall ties and straps

Wall ties should comply with BS1243[25] or meet the recommendations of DD140-2[26] for performance-designed ties. In situations of severe exposure, or where required by building regulations, suitable grade austenitic stainless steel or appropriate non ferrous ties should be used. The most frequently specified straps and ties are grade 304 austenitic stainless steel, although low carbon steel protected with a zinc coating to BS EN ISO 1461[27] or a minimum weight of coating $940g/m^2$ can be used in appropriate circumstances.

Guidance on the selection of walls ties, material and type, is given in Tables 6 and 10.

The materials and type of perimeter anchorages for laterally loaded wall panels should follow the principles described for cavity-wall ties when assessing durability requirements.

Anchorage straps for tying down roofs or similar should be 19 x 3mm galvanized mild steel or non-ferrous metal as appropriate to the recommended cavity ties.

Restraint straps for tying in walls should be 30 x 5mm galvanized mild steel, stainless steel, or non-ferrous metal as appropriate.

| **Table 5 Notional fire resistance of walls – loadbearing single-leaf walls** | | | | | | | |
|---|---|---|---|---|---|---|---|
| Masonry unit | Type | Minimum thickness (mm) for notional periods of fire resistance | | | | | |
| | | 4h | 3h | 2h | 90 mins | 60 mins | 30 mins |
| Clay brick | solid | 170 | 170 | 100 | 100 | 90 | 90 |
| | Not less than 75% solid, e.g. perforated | 200 | 200 | 170 | 170 | 170 | 100 |
| Concrete block | solid (dense) | – | – | 100 | 100 | 90 | 90 |
| | hollow (dense) | – | – | – | – | – | 190 |
| | solid (lightweight) | 150 | 140 | 100 | 100 | 90 | 90 |
| | hollow (lightweight) | – | – | 100 | 100 | 100 | 90 |
| | aerated | 180 | 140 | 100 | 100 | 90 | 90 |
| Concrete or calcium silicate brick | solid | 190 | 190 | 100 | 100 | 90 | 90 |

Notes
**a** Thickness can be reduced by approximately 10mm if not less than 13mm plaster or render is applied to each face (see Table 15 BS5628-3[2]).
**b** Non-loadbearing walls can have reduced thickness, especially for 90mins, 60mins and 30 mins periods (see Table 15 BS5628-3).
**c** Table 5 is much simplified from Table 15 of BS5628-3, and is to be used for initial design purposes only. Reference to BS5628-3 should be made to determine final minimum fire resistance requirements.

Metal connections, such as joist hangers or straps connecting the inner leaf with a buttressing partition which do not pass through the cavity to be embedded in the outer leaf of a cavity wall may be of galvanized mild steel, irrespective of the number of storeys in the building.

### 3.7 Damp-proof courses (dpcs)

Despite the widespread use of damp-proof courses in masonry elements, their structural properties, particularly in tension, have not been widely studied. Current British Standards do not define structural performance requirements.

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 strengths of dpcs are particularly suspect. Commonly used dpc materials and their properties are listed in Table 7.

| **Table 6 Recommended minimum types and qualities of cavity wall ties** | |
|---|---|
| Types of wall and building | Minimum recommended ties |
| 2 storeys or less | |
| i) domestic buildings | 10g stainless steel or non-ferrous butterfly or double triangle |
| ii) non-domestic and non-agricultural | 10g stainless steel or non-ferrous metal butterfly or double triangle |
| Not exceeding 3 storeys | |
| i) cavity less than 75mm | 10g stainless steel or non-ferrous butterfly or double triangle |
| ii) cavity exceeding 75mm but less than 175mm | 19 x 3mm stainless steel[a] or non-ferrous metal vertical twist (use equivalent safety tie) |
| Exceeding 3 storeys | |
| iii) cavity less than 75mm | 10g stainless steel or non-ferrous butterfly or double triangle |
| iv) cavity exceeding 75mm but less than 175mm | 19 x 3mm stainless steel or non-ferrous metal vertical twist (use equivalent safety tie) |

Note
**a** All cavity wall ties should comply with BS1243[25] or be of equivalent characteristics or performance designed alternatives (see DD140-2[26]).

**Table 7 Physical properties and performance of materials for dpcs**

| Material | Minimum mass (kg/m$^2$) | Minimum thickness (mm) | Joint treatment to prevent water moving | | Liability to extrusion | Durability | Other considerations |
|---|---|---|---|---|---|---|---|
| | | | Upward | Downward | | | |
| Flexible lead sheet | Code No 4 in BS743[28] | 1.8mm | Min 100mm lap | 100mm passing lap and interlocking upstand | Not under pressure met in normal construction | Corrodes in contact with mortars, bitumen paint protection required | Easily worked but slow process. Limit lengths to 1.5m |
| Bitumen Hessian base (class A of BS6398[29]) | 3.8 | – | Lapped at least 100mm | Lapped at least 100mm and sealed | Likely to extrude under heat and moderate pressure but this is unlikely to affect resistance to moisture penetration | The hessian or fibre may decay but this does not affect efficiency if the bitumen remains undisturbed. Classes D, E and F are most suitable for buildings that are intended to have a very long life or where there is a risk of movement | Materials should be unrolled with care. In cold weather, warm before use. When used as a cavity tray, the dpc should be fully supported.<br><br>For further guidance see BS6398 Annex B |
| Bitumen Fibre base (class B of BS6398) | 3.3 | – | | | | | |
| Bitumen Hessian base and lead (class A of BS6398) | 4.4 | – | | | | | |
| Bitumen Fibre base and lead (class E of BS6398) | 4.4 | – | | | | | |

Notes

**a** Table 7 is a condensed summary extract from Table 3 of BS5628–3[2].

**b** Other materials that have a British Board of Agrément certificate are in common use as dpcs, e.g. pitch polymer, bitumen polymer and polyethylene.

# 4 General principles of limit-state design for masonry walls and columns

## 4.1 Loadings

This *Manual* adopts limit-state principles and the partial factor format of BS5628-1[1]. The loads to be used in calculations are therefore:

(a) characteristic dead load, $G_k$: the weight of the structure complete with finishes, fixtures and fixed partitions (BS648[30] and BS6399-1[31])

(b) characteristic imposed load, $Q_k$: (BS6399-1 and -3[32] and the appropriate Building Regulations[33])

(c) characteristic wind load, $W_k$: (BS6399-2[34])

(d) at the ultimate limit state the building should be capable of resisting a uniformly distributed horizontal load equal to:
- 1.5% of the total characteristic dead load above any level (refer to 5.3.1)

(e) for the design of structural members affording horizontal or vertical lateral support to the masonry elements, including the elements transmitting this force to the members providing stability to the whole structure, the sum of:
- the simple static reaction arising from the total design horizontal forces applied at the lateral support, and
- 2.5% of the total vertical design load, applied as a horizontal force at the lateral support.

The horizontal force produced by (d) should be distributed between the strongpoints providing overall lateral stability, according to their stiffnesses. The strongpoints do not need to be designed to resist the horizontal force produced by (e).

The design loads are obtained by multiplying the characteristic loads by the appropriate partial safety factor, $\gamma_f$, from Table 8. The ‘adverse’ and ‘beneficial’ factors should be used so as to produce the most onerous condition.

**Table 8 Partial safety factors for loads,** $\gamma_f$ (Excluding accidental damage)

| Load combination | Load type | | | | |
|---|---|---|---|---|---|
| | dead $G_k$ | | imposed $Q_k$ | | wind $W_k$ |
| | adverse | beneficial | adverse | beneficial | |
| 1. Dead and imposed | 1.4 | 0.9 | 1.6 | 0 | – |
| 2. Dead and wind | 1.4 | 0.9 | – | – | 1.4[a] |
| 3. Dead, wind and imposed | 1.2 | – | 1.2 | – | 1.2 |

Note

**a** For infill panels subject to lateral wind loading only, a factor of 1.2 may be used where removal of the wall will in no way affect the stability of the remaining structure.

## 4.2 Serviceability limit state

No calculations are required to check the serviceability limit states of masonry elements, provided that the recommendations of this *Manual* are observed.

## 4.3 Characteristic strengths

### 4.3.1 Characteristic compressive strength

The characteristic compressive strength, $f_k$, for brick walls is given in Fig. 13. For brick walls or loaded inner leaf of a cavity wall where the thickness of the wall is equal to the width of a standard format brick, i.e. 103mm, the value given in Fig. 13 may be multiplied by 1.15.

The characteristic compressive strength $f_k$, for blockwork walls constructed of standard format blocks of 100mm, 140mm, and 215mm width, with a height of 215mm, is given in Figs 14, 15 and 16, respectively. For walls constructed with solid and hollow blocks of different sizes, reference should be made to BS5628-1[1], Table 2.

For walls or columns with plan area less than 0.2m$^2$, the characteristic compressive strength from Figs 13 to 16 should be multiplied by $(0.7 + 1.5A)$, where $A$ is the loaded cross-sectional plan area in m$^2$.

For hollow or perforated masonry units the characteristic compressive strength quoted when tested in accordance with the appropriate British Standard relates to the gross plan area of the masonry unit as though it was solid.

Where hollow blocks are filled with *in-situ* concrete with a compressive strength greater than the compressive strength of the block, recalculated using the net area of the block, the characteristic compressive strength of the masonry may be determined from Figs 14 to 16, assuming that the blocks are solid and have a compressive strength as recalculated. Where the compressive strength of the concrete infill is less than the recalculated compressive strength of the block, the characteristic compressive strength of the masonry should be determined on the basis of the compressive strength of the concrete infill.

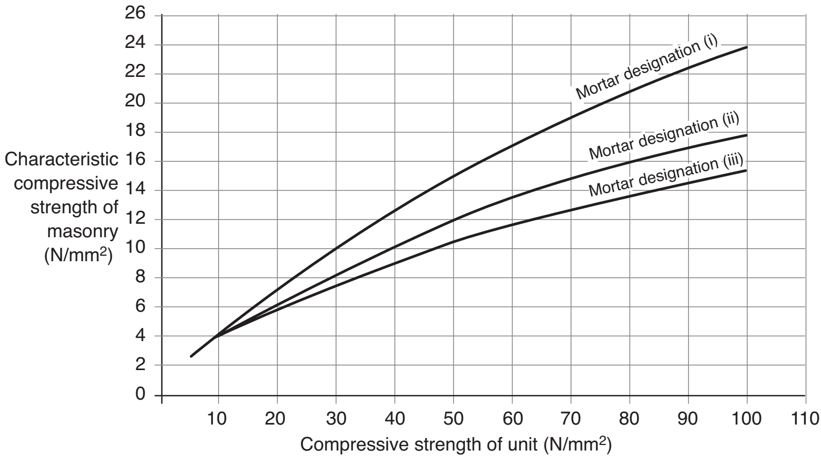



**Fig. 13 Characteristic compressive strength of brick masonry, $f_k$**

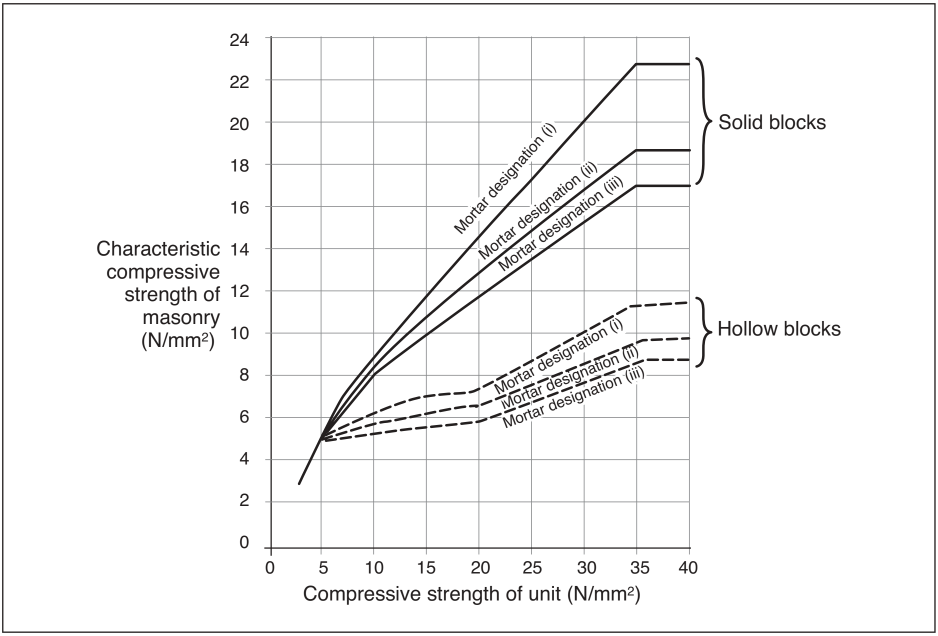



**Fig. 14 Characteristic compressive strength, $f_k$, for blockwork walls built with standard format blocks 100mm wide and 215mm high**

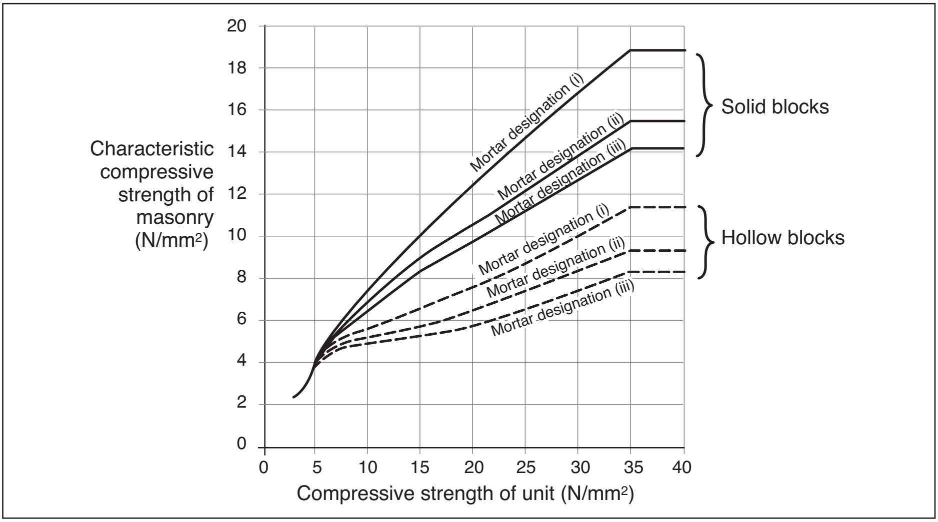



**Fig. 15 Characteristic compressive strength, $f_k$, for blockwork walls built with standard format blocks 140mm wide and 215mm high**

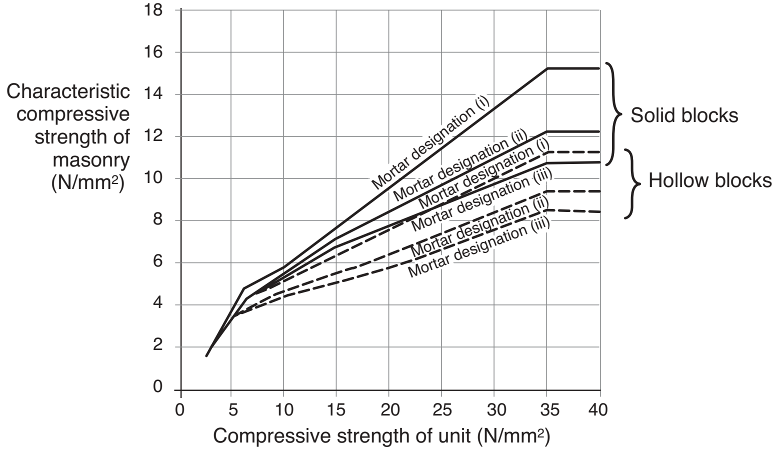



**Fig. 16 Characteristic compressive strength, $f_k$, for blockwork walls built with standard format blocks 215mm wide and 215mm high**

For masonry units laid other than on their bed face, the compressive strength of the units in that direction should be used to determine $f_k$ using Table 2 in BS5628-1[1]. When using 100mm x 215mm wide solid blocks laid flat refer to Table 2(b).

If the blocks are to be laid with mortar only on the outer surfaces of the blocks (shell bedding) the design strength needs to be reduced by the ratio of the bedded area to the gross area of the block. Shell bedding is not recommended for structural masonry.

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 concentrated load may be assumed to be uniformly distributed over the area of the bearing, except in the special case of a spreader located at the end of a wall and spanning in its plane (bearing type 3, see Fig. 17c) and dispersed in two planes within a zone contained by lines extending downwards at 45$^o$ from the edges of the loaded area.

The effect of the local load combined with stresses from other loads (see Fig. 18a) should be checked:

(a) at the bearing, assuming a local bearing design strength of 1.25 $f_k/\gamma_m$ in the case of bearing type 1 (Fig. 17a) or 1.5 $f_k/\gamma_m$ in the case of bearing type 2 (Fig. 17b)

(b) at a distance of 0.4$h$ below the bearing where the design strength is to be taken as $\beta f_k/\gamma_m$ allowing for the effects of slenderness

where

$f_k$ is the characteristic compressive strength of the masonry

$h$ is the clear height of the wall

$\gamma_m$ is the appropriate partial safety factor for material strength

$\beta$ is the capacity reduction factor for slenderness (see Table 14)

In the special case of a spreader beam, designed in accordance with an acceptable elastic theory, located at the end of a wall and spanning in its plane (see Fig. 17c), the maximum stress at the bearing combined with stresses due to other loads should not exceed 2.0 $f_k / \gamma_m$ (see Fig. 18b). At the distance of 0.4*h* below the bearing, the design strength should be taken as $\beta \; f_k / \gamma_m$ allowing for the effects of slenderness.

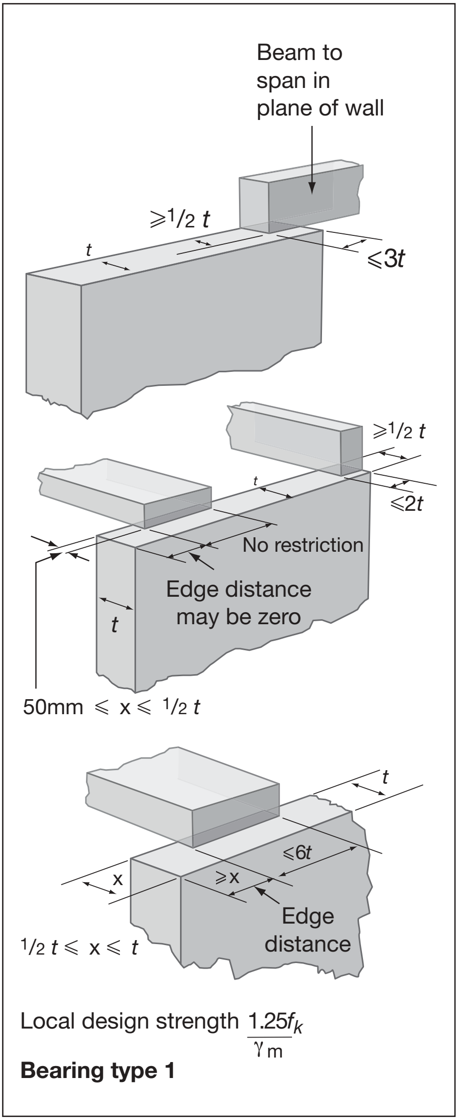



**Fig. 17a Concentrated loads: bearing type 1**

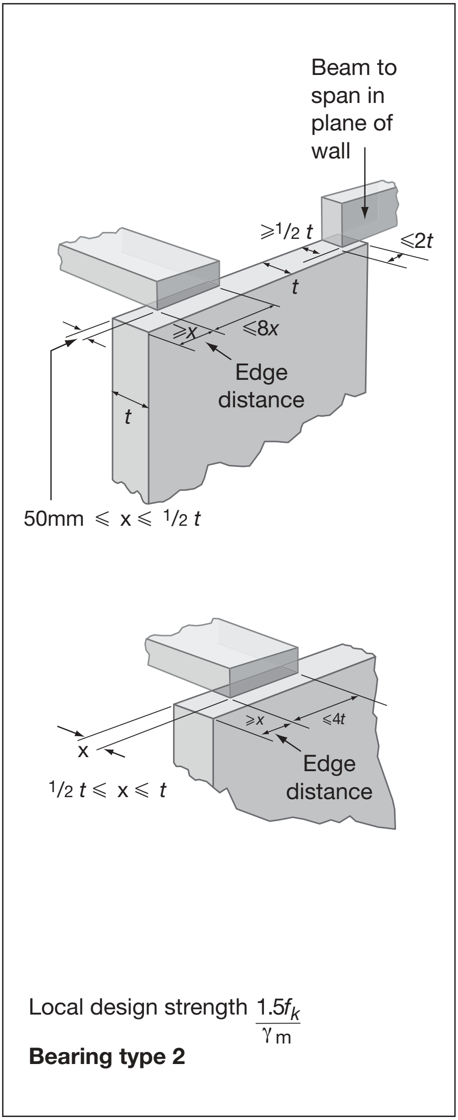
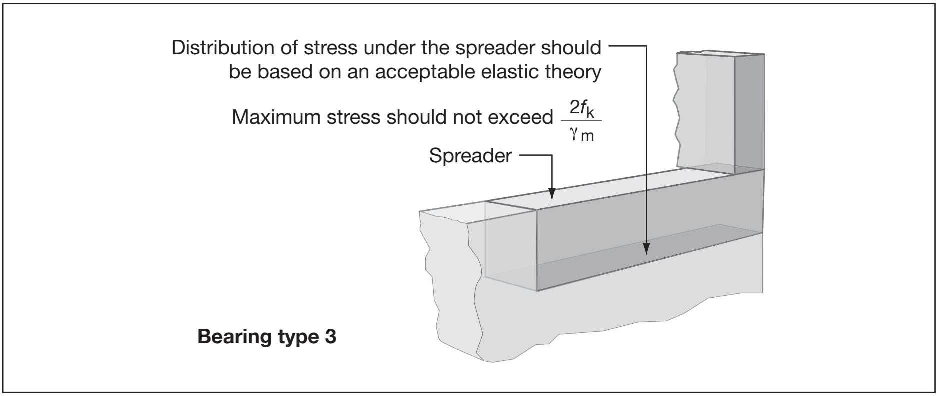
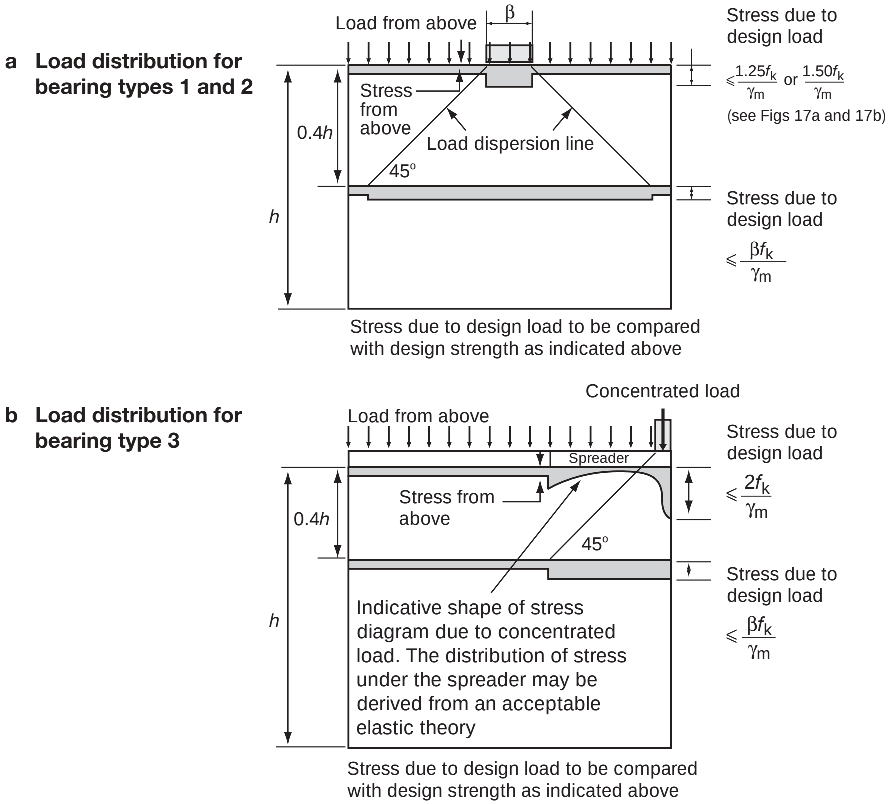



**Fig. 17b Concentrated loads: bearing type 2**




## Fig. 17c Concentrated loads: bearing type 3




## Fig. 18 Concentrated loads: load distribution

### 4.3.2 Characteristic flexural strength

Suggested values of the characteristic flexural strength $f_{kx}$ of normally bonded masonry are given in Table 9. In general direct tension should not be allowed.

### 4.3.3 Characteristic shear strength

The characteristic shear strength of masonry, $f_v$, in the horizontal direction of the horizontal plane (Fig. 19) may be taken as $0.35 + 0.6g_A$ N/mm$^2$ with a maximum of 1.75N/mm$^2$ for walls built in mortar designations (i) and (ii) or $0.15 + 0.6g_A$ N/mm$^2$ with a maximum of 1.4N/mm$^2$ for walls built in mortar designations (iii) and (iv) where $g_A$ is the design vertical load per unit area of wall cross-section due to the vertical loads calculated from the appropriate loading condition.

The characteristic shear strength $f_v$, of bonded masonry in the vertical direction of the vertical plane (Fig. 19) may be taken as:

(a) for brick:
0.7N/mm$^2$ for mortar designations (i) and (ii);
0.5N/mm$^2$ for mortar designation (iii), and (iv)

(b) for dense aggregate solid concrete block with a minimum strength of 7N/mm$^2$:
0.35N/mm$^2$ for mortar designations (i), (ii) and (iii).

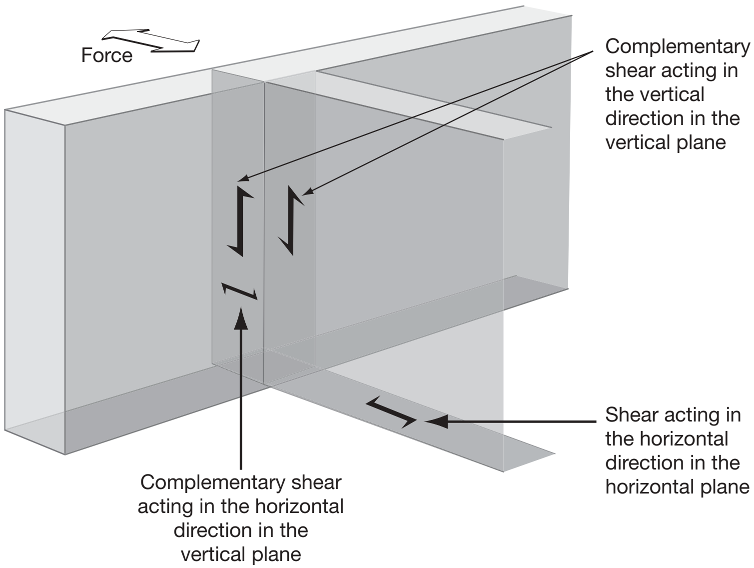
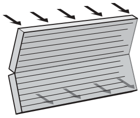
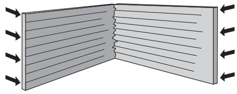



**Fig. 19 Shear forces acting in the horizontal and vertical planes**

## Table 9 Characteristic flexural strength of masonry, $f_{kx}$ (N/mm²)

| | | Plane of failure parallel to bed joints | | | Plane of failure perpendicular to bed joints | | |
|---|---|---|---|---|---|---|---|
| | |  | | |  | | |
| Mortar designation | | (i) | (ii) and (iii) | (iv) | (i) | (ii) and (iii) | (iv) |
| Clay bricks having a water absorption | | | | | | | |
| less than 7% | | 0.7 | 0.5 | 0.4 | 2.0 | 1.5 | 1.2 |
| between 7% and 12% | | 0.5 | 0.4 | 0.3 | 1.5 | 1.1 | 1.0 |
| over 12% | | 0.4 | 0.35 | 0.25 | 1.1 | 0.9 | 0.8 |
| Calcium silicate bricks | | 0.3 | | 0.2 | 0.9 | | 0.6 |
| Concrete bricks | | 0.3 | | 0.2 | 0.9 | | 0.6 |
| Concrete blocks (solid or hollow) of compressive strength in N/mm² | | | | | | | |
| 2.8 | Used in walls of thickness up to 100mm[a] | 0.25 | | 0.2 | 0.40 | | 0.4 |
| 3.5 | | | | | 0.45 | | 0.4 |
| 7.0 | | | | | 0.60 | | 0.4 |
| 2.8 | Used in walls of thickness up to 125mm[a] | 0.233 | | 0.183 | 0.375 | | 0.367 |
| 3.5 | | | | | 0.417 | | 0.367 |
| 7.0 | | | | | 0.558 | | 0.467 |
| 2.8 | Used in walls of thickness up to 140mm[a] | 0.223 | | 0.173 | 0.360 | | 0.347 |
| 3.5 | | | | | 0.397 | | 0.347 |
| 7.0 | | | | | 0.533 | | 0.447 |
| 2.8 | Used in walls of thickness up to 190mm[a] | 0.19 | | 0.14 | 0.31 | | 0.28 |
| 3.5 | | | | | 0.33 | | 0.28 |
| 7.0 | | | | | 0.45 | | 0.38 |
| 2.8 | Used in walls of thickness up to 215mm[a] | 0.173 | | 0.123 | 0.285 | | 0.247 |
| 3.5 | | | | | 0.297 | | 0.247 |
| 7.0 | | | | | 0.408 | | 0.347 |
| 2.8 | Used in walls of thickness up to 250mm[a] | 0.15 | | 0.1 | 0.25 | | 0.2 |
| 3.5 | | | | | 0.25 | | 0.2 |
| 7.0 | | | | | 0.35 | | 0.3 |
| 10.5 | Used in walls of any thickness[a] | 0.25 | | 0.2 | 0.75 | | 0.6 |
| 14.0 | | | | | 0.90[b] | | 0.7[b] |

Notes

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

**b** When used with flexural strength in parallel direction, assume the orthogonal ratio $\mu = 0.3$

**c** Linear interpolation is allowed for concrete block walls of thickness between 100mm and 250mm and concrete blocks of compressive strength between 2.8 and 7.0N/mm² in a wall of given thickness.

**d** Flexural tensile stresses should generally not be allowed at damp-proof courses except where brick or slate is used as the dpc, but partial fixity may be provided due to the action of dead loads (see subsection 5.4.4).

### 4.3.4 Characteristic strength of wall ties

The characteristic strengths of wall ties used to restrain wall panels subjected to lateral loads are given in Table 10, but to meet the design recommendations of DD140-2[26], they should be determined in accordance with DD140-1[35].

## 4.4 Design strength

The design strength is equal to the characteristic strength divided by the appropriate partial factor for material strength $\gamma_m$. Values of $\gamma_m$ for normal design loads are given in Table 11.

Unless the requirements of the special categories of manufacturing and construction control can be assured it is recommended that normal categories be assumed in design.

The categories are defined as follows:

*Manufacturing control*

'Normal category' should be assumed when the supplier of the masonry units is able to meet the requirements for compressive strength in the appropriate British Standard.

'Special category' may be assumed where the manufacturer:

(a) agrees to supply consignments of structural units to a specified 'acceptance limit' for compressive strength, such that the average compressive strength of a sample of the structural units, taken from any consignment and tested in accordance with the appropriate British Standard specification, has a probability of not more than 2.5% of being below the acceptance limit, and

(b) operates a quality control scheme, the results of which can be made available to demonstrate to the satisfaction of the purchaser that the acceptance limit is consistently being met in practice, with the probability of failing to meet the limit being never greater than that stated in (a).

*Construction control*

'Normal category' should be assumed whenever the work is carried out following the recommendations for workmanship in Annex A of BS5628-3[2] or BS8000-3[36], including appropriate supervision and inspection.

'Special category' may be assumed where the requirements of normal category control are complied with and in addition:

(a) the specification, supervision and control ensure that the construction is compatible with the use of the appropriate partial safety factors given in Table 11, and

(b) preliminary compressive strength tests carried out on the mortar, in accordance with Annex A1 of BS5628-1[1], indicate compliance with the strength requirements in Table 2 and regular testing of the mortar used on site in accordance with Annex A1 of BS5628-1 shows that compliance with the strength requirement given in Table 2 is being maintained.

**Table 10 Characteristic strengths of wall ties used as panel supports**

| Type | Characteristic strengths of ties engaged in dovetail slots set in structural concrete: Tension and compression | Characteristic strengths of ties engaged in dovetail slots set in structural concrete: Shear |
|---|---|---|
| Dovetail slot types of tie | kN | kN |
| (a) Galvanised or stainless steel fishtail[a] anchors 3mm thick, 17mm min. width in 1.25mm thick galvanised or stainless steel slot, 150mm long, set in structural concrete | 4.0 | 5.0 |
| (b) Galvanised or stainless steel fishtail[a] anchors 2mm thick, 17mm min. width in 2mm thick galvanised or stainless steel slots 150mm long, set in structural concrete | 3.0 | 4.5 |
| (c) Copper fishtail[a] anchors 3mm thick, 17mm min. width, in 1.25mm copper slots, 150mm long, set in structural concrete | 3.5 | 4.0 |

| | Characteristic loads in ties embedded in mortar | | | | | | |
|---|---|---|---|---|---|---|---|
| | Compression[b] | | | Tension | | | Shear[c] |
| | Mortar designation | | | Mortar designation | | | Mortar designation |
| | (i) and (ii) | (iii) | (iv) | (i) and (ii) | (iii) | (iv) | (i), (ii) or (iii) |
| Cavity wall ties[d] | kN | kN | kN | kN | kN | kN | kN |
| (a) Wire butterfly type: Zinc coated mild steel or stainless steel | 0.5 (0.35) | 0.5 (0.35) | 0.5 (0.35) | 3.0 | 2.5 | 2.0 | 2.0 |
| (b) Vertical twist type: Zinc coated mild steel, bronze or stainless steel | 5.0 (4.5) | 4.0 (3.5) | 2.5 (2.5) | 5.0 | 4.0 | 2.5 | 3.5 |
| (c) Double triangle type: Zinc coated mild steel | 1.25 (0.65) | 1.25 (0.65) | 1.25 (0.65) | 5.0 | 4.0 | 2.5 | 3.0 |

Notes

**a** Use equivalent safety ties.

**b** Maximum cavity width 75mm. Values for 100mm cavity shown in brackets.

**c** Applicable only to cases where shear exists between closely abutting surfaces.

**d** Ties in accordance with BS1243[25].

**e** The gap between wall and supporting structure is to be not greater than 75mm. Where cavities exceed 75mm, the compressive resistance of the ties should be determined in accordance with DD140-1[35]. The tie manufacturers can generally provide this information.

**f** Values for wire butterfly and double triangle type wall ties applicable only if the requirements in subsection 3.6 are met.

**Table 11 Partial safety factors for material strength $\gamma_m$ for normal design loads**

| Material strength | Category of construction control | | Category of manufacturing control |
|---|---|---|---|
| | Special | Normal | |
| Masonry Compression | 2.5<br>2.8 | 3.1<br>3.5 | Special<br>Normal |
| Masonry Flexure | 2.5 | 3.0 | |
| Masonry Shear $\gamma_{mv}$ | 2.5 | 2.5 | |
| Wall ties | 3.0 | 3.0 | |

# 5 Design of loadbearing masonry

## 5.1 Load combinations

Load combinations for masonry design, excluding accidental damage, are given in Table 8. Strongpoints, if constructed from masonry, need to be checked for all three load combinations. Other elements should be checked for either load combinations 1 or 2. Usually masonry elements supporting large vertical loads (e.g. walls in the lower storeys of a multistorey building) need to be checked for load combination 1 only. For masonry elements supporting small vertical loads, load combination 2 will usually be critical (e.g. walls in the top storey of a multistorey building, or walls in a single storey building).

## 5.2 Design procedure

The normal design procedure for loadbearing masonry is:

(a) consider overall stability and check that strongpoints are sufficient to resist horizontal loading and that floors and roof can act as horizontal diaphragms to transfer lateral loads into strongpoints
(b) consider robustness (see section 2.7)
(c) determine minimum requirements of unit quality and mortar strength for durability (see chapter 3)
(d) determine requirements for minimum thicknesses of members for fire (see section 3.5)
(e) check the architect's requirements for such matters as thermal value, sound transmission, aesthetics, durability, dpcs and partitions
(f) 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 that points of lateral support and any anchorages assumed in the calculations can be achieved in practice, together with the effects of any dpcs (particularly narrow piers between windows), services perforations and joints.
(g) make calculations
(h) prepare details and specifications, and include provision for movement both of walling elements and of the overall building.

## 5.3 Walls and piers subject to vertical load

The design of vertical loadbearing masonry is based on consideration of slenderness and buckling. The end restraint conditions of the masonry elements are therefore important.

### 5.3.1 Lateral supports

A lateral support may be provided along either a horizontal or a vertical line, depending on whether the slenderness ratio is based on a vertical or horizontal dimension.

*Horizontal or vertical supports*

Vertical lateral supports, (e.g. buttressing walls) and horizontal lateral supports (e.g. floors or roofs acting as horizontal girders) should be capable of transmitting, to those elements of construction that provide lateral stability to the structure as a whole (termed 'strongpoints'), the sum of the following design lateral forces:

(a) the simple static reactions to the total applied design horizontal forces (e.g. wind loads) at the lateral support, and
(b) 2.5% of the total design vertical load that the wall or column is designed to carry at the line of lateral support.

In the majority of cases the strongpoints need to be designed only for the larger of the two, which would normally be (b). There are conditions, however, (e.g. ground storey of podium construction; two-storey building and long-span heavily-loaded first floor) where the designer may consider it more appropriate for the strongpoints also to be designed to resist both (a) and (b) above.

It should be noted that BS5628-1[1] requires 1.5% of the total characteristic dead load ($0.15G_k$) above any level to be resisted by the structure as a whole. This is not considered to be adequate, and thus it is recommended that 2.5% of the total characteristic dead load is used for this purpose.

Figs 20 to 22 illustrate connections that may be used to provide simple lateral restraint. In the illustrations floors are generally shown: however, similar details are applicable to roofs. The effective cross-section of anchors and of their fixings should be capable of resisting the design loads as noted above, assuming a design strength equal to the characteristic yield strength (or its equivalent) as laid down in the appropriate British Standard divided by $\gamma_m = 1.15$. Anchors should be provided at intervals of not more than 2m in houses of not more than 3 storeys and not more than 1.25m for storeys in all other buildings. Galvanized mild-steel anchors having a cross-section of 30 x 5mm may be assumed to have adequate strength in buildings of up to 6 storeys in height. All straps and fittings should be of galvanized or stainless steel.

### *Simple horizontal restraints*

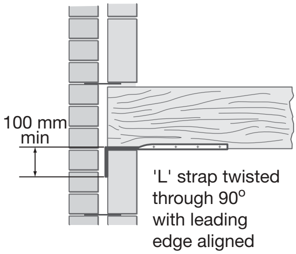
Simple resistance to lateral movement may be assumed in the case of houses of not more than 3 storeys where timber floor members, spaced apart at a distance of not more than 1.2m, are connected by suitable joist hangers effectively fixed to the joist. In all other cases, including buildings of more than 3 storeys, connections of the form illustrated in Figs 20, 21 and 22 (based on BS5628-1[1] and -3[2]) will usually provide simple resistance to lateral movement.

### *Enhanced horizontal restraints*

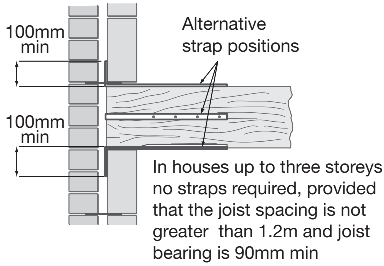
Enhanced resistance to lateral movement may be assumed where:

- floors or roofs of any form of construction span on to the wall or column from both sides at the same level, which if positioned at the top of the wall are positively fixed to the wall with straps on both sides of the wall
- an *in-situ* concrete floor or roof, or a precast concrete floor or roof giving equivalent restraint, irrespective of the direction of span, has a bearing of at least one-half the thickness of the wall or inner leaf of a cavity wall or column on to which it spans, but in no case is less than 90mm
- in the case of houses of not more than 3 storeys, a timber floor spans on to a wall from one side and has a bearing of not less than 90mm.

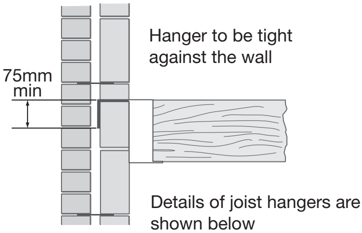
  Preferably, columns should be provided with lateral support in both horizontal directions.



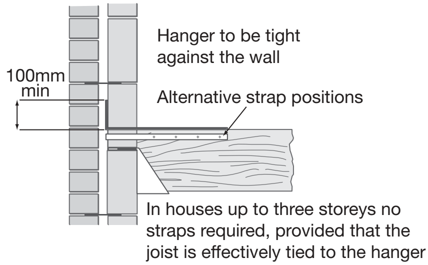

**Timber floor bearing directly onto wall**



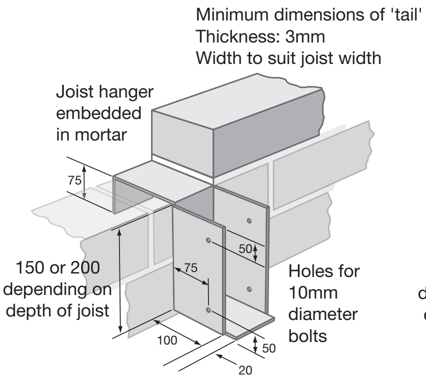

**Timber floor bearing directly onto wall**

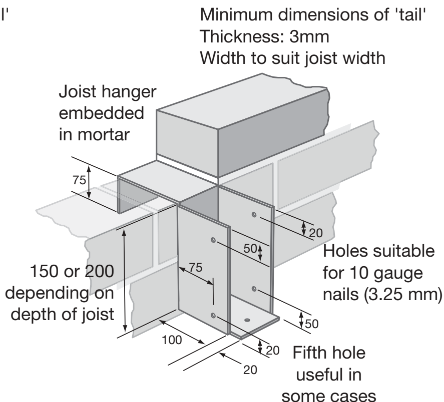



**Timber floor using nailed or bolted joist hangers acting as tie**




**Timber floor using typical joist hanger**



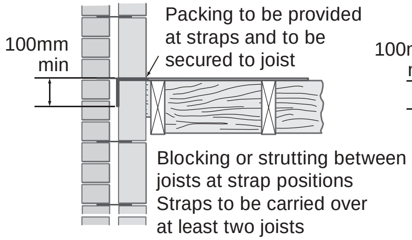




*All dimensions are in millimetres*

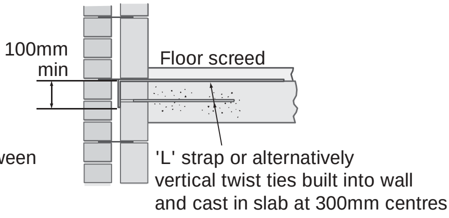
**Joist hanger as tie: bolted form**

**Joist hanger as tie: nailed form**

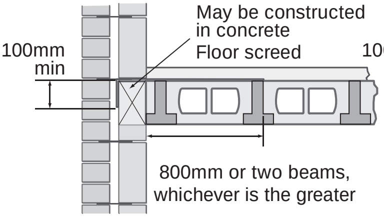
**Fig. 20 Connections that may be used to provide simple lateral restraint**



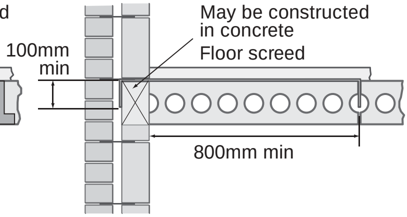

**Timber floor abutting external cavity wall**




*** In situ concrete floor abutting external cavity wall**

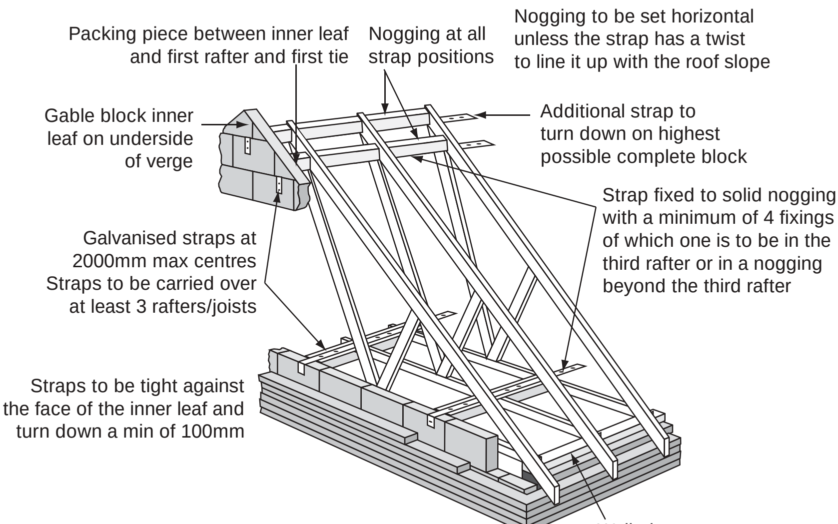



*** Beam and pot floor abutting external cavity wall**




*** Precast units abutting external cavity wall**

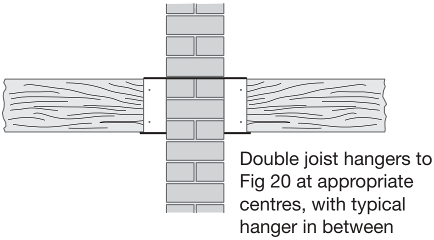
*Deflection of floor at junction with wall could cause screed to crack at strap locations



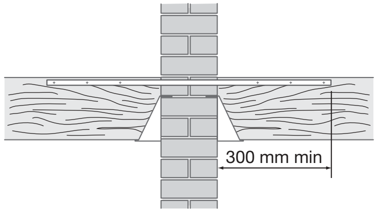

**Truss roofs with straps**

Tiling battens omitted for clarity

**Fig. 21**

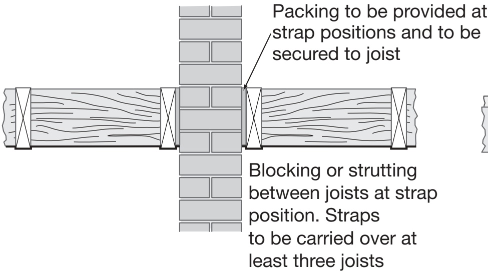



**Timber floor using double joist hanger acting as tie**

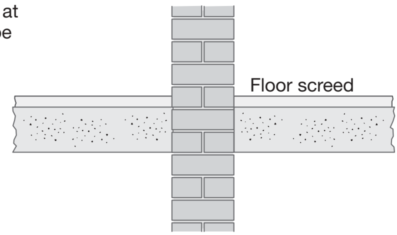



In houses up to three storeys no straps are required, provided that the joist is effectively fixed to the hanger. Such fixing can be assumed if joist hangers to Fig 20 are provided at no more than 1.2m centres, with typical hangers in between

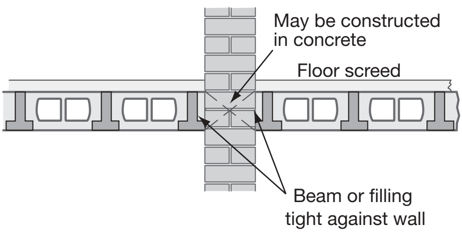
**Timber floor using double joist hanger**



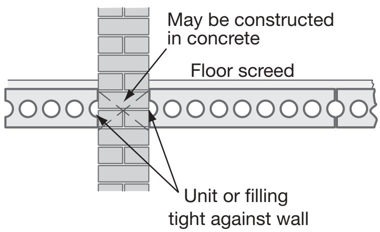

**Timber floor abutting internal wall**




**In-situ floor abutting internal wall**




**Beam and pot floor abutting internal wall**




**Precast units abutting internal wall**

**Fig. 22**

### *Design of horizontal restraints*

The engineer should not assume that floors, roofs, etc. provide adequate restraint to a wall. The engineer should check that the restraining element is capable of transferring the load to the elements providing stability to the building. Particular attention is drawn to connections and the use of trussed rafters, where purpose-designed bracing is often required. The engineer must consider loads in both directions.

### *Vertical lateral supports*

Simple resistance to lateral movement may be assumed where an intersecting or return wall not less than the thickness of the supported wall or loadbearing leaf of a cavity wall extends from the intersection at least 10 times the thickness of the supported wall or loadbearing leaf and is connected to it by metal ties evenly distributed throughout the height at not more than 300mm centres, and capable of resisting the forces defined above. The strengths of various types of ties are shown in Table 10.

Enhanced resistance to lateral movement may be assumed where an intersecting or return wall as described above is properly bonded to the supported wall or loadbearing leaf of a cavity wall.

In all other cases of vertical lateral support, simple or enhanced resistance to lateral movement may be established by calculation.

## 5.3.2 Effective height

### *Walls*

The effective height of a wall may be taken as:

- 0.75 times the clear distance between lateral supports that provide enhanced resistance to lateral movement, or
- the clear distance between lateral supports that provide simple resistance to lateral movement.

Particular consideration should be given to the effective height of panels to the first effective line of restraint.

### *Columns*

A column is an isolated vertical loadbearing member whose width is not more than 4 times its thickness. The effective height of a column should be taken as the distance between lateral supports or twice the height of the column in respect of a direction in which lateral support is not provided.

### *Columns formed by adjacent openings in walls*

Where openings occur in a wall such that the masonry between any two openings is, by definition, a column, the effective height of the column should be taken as follows:

- where an enhanced resistance to lateral movement of the wall containing the column is provided, the effective height should be taken as 0.75 times the distance between the supports plus 0.25 times the height of the taller of the two openings
- where a simple resistance to lateral movement of the wall containing the column is provided, the effective height should be taken as the distance between the supports.

*Piers*

Where the thickness of a pier is not greater than 1.5 times the thickness of the wall of which it forms a part, it may be treated as a wall for effective height consideration; otherwise the pier should be treated as a column in the plane at right angles to the wall.

It should be noted that the thickness of a pier, $t_p$, is the overall thickness including the thickness of the wall or, when bonded into one leaf of a cavity wall, the thickness obtained by treating this leaf as an independent wall.

### 5.3.3 Effective length

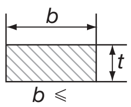
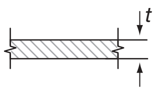
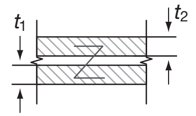
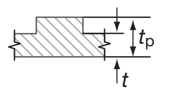
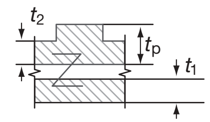
The effective length of a wall may be taken as:

- 0.75 times the clear distance between vertical lateral supports or twice the distance between a support and a free edge, where lateral supports provide enhanced resistance to lateral movement
- the clear distance between lateral supports or 2.5 times the distance between a support and a free edge where lateral supports provide simple resistance to lateral movement.

### 5.3.4 Effective thickness

The effective thickness, $t_{ef}$, of a wall, a wall stiffened by piers and a column is shown in Table 12.

**Table 12 Effective thickness of columns and walls**

| Column | Single-leaf wall | Cavity wall | Wall stiffened by piers | |
|---|---|---|---|---|
| | | | Single leaf | Cavity |
| Plan shapes | | | | |
|  |  |  |  |  |
| Effective thickness $t_{eff}$ | | | | |
| $t$ or $b$, depending on direction of bending | $t$ | the greatest of<br>(a) $\frac{2}{3}(t_1 + t_2)$<br>or<br>(b) $t_1$<br>or<br>(c) $t_2$ | $tK$ | the greatest of<br>(a) $\frac{2}{3}(t_1 + Kt_2)$<br>or<br>(b) $t_1$<br>or<br>(c) $Kt_2$ |
| | | | where $K$ is a coefficient from Table 13 | |

When determining the effective thickness of a wall stiffened by intersecting walls, the appropriate stiffness coefficient may be determined from Table 13 on the assumption that the intersecting walls are equivalent to piers of width equal to the thickness of the intersecting wall and of thickness equal to 3 times the thickness of the stiffened wall.

From the geometric properties of a diaphragm wall ($I$ = second moment of area: $A$= area: $r$ = radius of gyration) its effective thickness can be shown to be equivalent to that of a solid wall of greater thickness. This confirms the efficiency of such walls in their axial load carrying capacity. However, in design the effective thickness of a diaphragm wall is usually taken as the actual thickness.

### 5.3.5 Slenderness ratio

The slenderness ratio is the ratio of the effective height or the effective length to the effective thickness. It should generally not exceed 27, but for walls less than 90mm thick in buildings of more than 2 storeys it should not exceed 20.

The slenderness ratios of 27 and 20 apply only to walls carrying an imposed vertical load and can be exceeded for a laterally loaded wall.

### 5.3.6 Eccentricity at right-angles to the wall

It may be assumed that the load transmitted to a wall by a single floor or roof acts at one third of the depth of the bearing area from the loaded face of the wall or loadbearing leaf. Where a uniform floor is continuous over a wall, each side of the floor may be taken as being supported individually on half the total bearing area. Where joist hangers are used, the load should be assumed to be applied at the face of the wall.

The resultant eccentricity of the load at any level may be calculated on the assumption that the resultant of all vertical loads immediately above a lateral support is axial.

### 5.3.7 Vertical load resistance

The design should be carried out using the principles given in subsection 5.3.8.

**Table 13 Stiffness coefficients *K* for walls stiffened by piers or intersecting walls**

| Ratio of pier spacing (centre to centre) to pier width | Ratio $t_p/t$ or $t_p/t_2$ 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 Table 13 is permissible, but not extrapolation outside the limits given.

### 5.3.8 Vertical load resistance of solid walls and columns

The design vertical load resistance per unit length of a single-leaf wall, which may or may not be stiffened by piers, is:

$$\beta \frac{t f_k}{\gamma_m}$$

where

$\beta$ is a capacity reduction factor allowing for the effects of slenderness and eccentricity and is obtained from Table 14

$\gamma_m$ is the appropriate partial safety factor for the material

$f_k$ is the characteristic strength of the masonry

$t$ is the thickness of the wall

The design vertical load resistance of a solid rectangular column is given by:

$$\beta \frac{b t f_k}{\gamma_m}$$

where

$b$ is the width of the column

$t$ is the thickness of the column

and all other symbols are as given above.

**Table 14 Capacity reduction factor, $\beta$**

| Slenderness ratio $h_{ef}/t_{ef}$ | Eccentricity at top of wall, $e_x$ | | | |
|---|---|---|---|---|
| | Up to 0.05$t$ (see note 1) | 0.1$t$ | 0.2$t$ | 0.3$t$ |
| 0 | 1.00 | 0.88 | 0.66 | 0.44 |
| 6 | 1.00 | 0.88 | 0.66 | 0.44 |
| 8 | 1.00 | 0.88 | 0.66 | 0.44 |
| 10 | 0.97 | 0.88 | 0.66 | 0.44 |
| 12 | 0.93 | 0.87 | 0.66 | 0.44 |
| 14 | 0.89 | 0.83 | 0.66 | 0.44 |
| 16 | 0.83 | 0.77 | 0.64 | 0.44 |
| 18 | 0.77 | 0.70 | 0.57 | 0.44 |
| 20 | 0.70 | 0.64 | 0.51 | 0.37 |
| 22 | 0.62 | 0.56 | 0.43 | 0.30 |
| 24 | 0.53 | 0.47 | 0.34 | |
| 26 | 0.45 | 0.38 | | |
| 27 | 0.40 | 0.33 | | |

Notes

**a** It is not necessary to consider the effects of eccentricities up to and including 0.05$t$.

**b** Linear interpolation between eccentricities and slenderness ratio is permitted.

**c** The derivation of $\beta$ is given in Appendix B of BS5628-1[1].

The value of β should be as follows:

- when the eccentricities about the major and minor axes are less than 0.05*b* but greater than 0.05*t*, respectively: from Table 14, using the values of eccentricity and slenderness ratio appropriate to the minor axis
- when the eccentricities about the major and minor axes are greater than 0.05*b* but less than 0.05*t*, respectively: from Table 14, using the value of eccentricity appropriate to the major axis and the value of slenderness ratio appropriate to the minor axis.
- when the eccentricities about the major and minor axes are greater than 0.05*b* but less than 0.05*t*, respectively: from Table 14, using the value of eccentricity appropriate to the major axis and the value of slenderness ratio appropriate to the minor axis.

For eccentricities about the major and minor axes greater than 0.05*b* and 0.05*t*, respectively: calculate the additional eccentricities using BS5628-1[1], Appendix B.

### 5.3.9 Vertical load resistance of cavity walls and columns

Where in a cavity wall the load is carried by one leaf only, the loadbearing capacity of the wall should be based on the horizontal cross-sectional area of that leaf alone, although the stiffening effect of the other leaf can be taken into account when calculating the slenderness ratio.

When the applied vertical load acts between the centroids of the two leaves of a cavity wall or column, it should be replaced by statically equivalent axial loads in the two leaves. Each leaf should then be designed to resist these calculated axial loads again taking into account the stiffening effect of the other leaf.

### 5.3.10 Eccentricity in the plane of the wall and shear wall

The eccentricity in the plane of a single wall can be calculated from statics alone (Fig. 23). Where a horizontal force is resisted by several walls it may be distributed between the walls in proportion to their flexural stiffnesses about an axis perpendicular to the direction of the force.

The forces in the walls may be determined by an appropriate elastic analysis. Connections for transmitting the horizontal force to the walls should be properly designed.

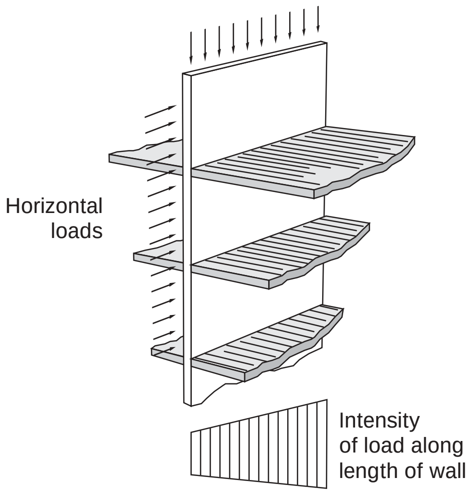



**Fig. 23 Load distribution from loading eccentric to plane of wall**

### 5.3.11 Horizontal shear resistance

The resistance to horizontal in plane shear forces may be assumed to be adequate if the following relationship is satisfied:

$$V_h < \frac{f_v}{\gamma_{mv}}$$

where

$V_h$ is the shear stress produced by the horizontal design load calculated as acting uniformly over the horizontal cross-sectional area of the wall

$\gamma_{mv}$ is the partial safety factor for material strength in shear

$f_v$ is the characteristic shear strength of the masonry (see subsection 4.3.3)

## 5.4 Walls subject to lateral loading

Walls subject to lateral loading may be designed either by using the wall panel bending moment coefficients given in Table 15 for 2-way spanning walls or by following the analysis guidance given in subsections 5.4.4 and 5.4.6 for 1-way spanning walls. An extended set of tables is available in *Concrete Masonry Designer's Handbook*[37].

The design moment of resistance for laterally loaded walls is given by:

$$\frac{f_{kx} Z}{\gamma_m}$$

where

$f_{kx}$ is the characteristic flexural strength appropriate to the plane of bending (i.e. $f_{kx1}$ or $f_{kx2}$ from Table 9)

$\gamma_m$ is the appropriate partial safety factor for materials

$Z$ is the section modulus of the wall profile

### 5.4.1 Limits on wall panel sizes (to be applied when using Table 15)

For laterally loaded panels in mortar designations (i) to (iv) dimensions should not exceed the following:

(a) Panel supported on three edges:
- (i) 2 or more sides continuous: height x length $\leqslant 1500t_{ef}^2$
- (ii) all other cases: height x length $\leqslant 1350t_{ef}^2$

where $t_{ef}$ is the effective thickness

(b) Panel supported on all four edges:
- (i) 3 or 4 sides continuous: height x length $\leqslant 2250t_{ef}^2$
- (ii) all other cases: height x length $\leqslant 2025t_{ef}^2$

In (a) and (b) height or length $50t_{ef}$

(c) Panel simply supported top and bottom height $\leqslant 40t_{ef}$ where the effective thickness is defined as for vertically loaded walls.

Examples of continuous and simple edge conditions are shown in Fig. 24.

It should be noted that the $50t_{ef}$ and $40t_{ef}$ dimensions exceed the slenderness ratios quoted in subsection 5.3.5 for vertically loaded walls. (This is acceptable in the case of laterally loaded wall panel design).

### 5.4.2 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 wall units are commonly laid on their bed face then the two flexural strengths described above relate to ‘horizontally spanning’ ($f_{kx2}$) and ‘vertically spanning’ ($f_{kx1}$) walls respectively. It is usually more economic to span masonry walls horizontally. Values of $f_{kx1}$ and $f_{kx2}$ may be obtained from Table 9.

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 subsection 5.4.3

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 return walls or columns may be fully bonded, tied or untied. Examples of such junctions and how they should be considered are shown in Fig. 24. In a cavity wall, continuity across a vertical junction may be assumed even if only one leaf is continuous over the column, 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 Fig. 24.

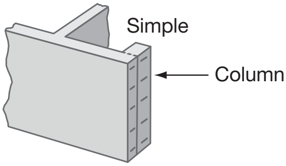
The effects of dpcs need to be considered in laterally loaded masonry. Their presence complicates the design since they generally act as a discontinuity in a laterally loaded wall. Some continuity is, however, still possible because of gravity structural action, which will be considered later. Otherwise, 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.

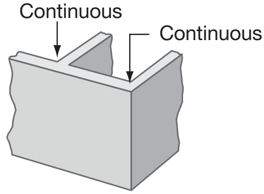
Two vertical movement joints acting as free edges result in the panel spanning vertically, i.e. in its weaker direction. It is more efficient to use the horizontal spanning capability of masonry, and therefore it is preferable to position the vertical movement joints at vertical lateral support locations.

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.

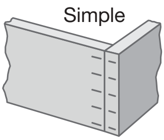
Consideration should be given to the ability of wall ties to transmit any out of balance tensile and compressive forces across the cavity (see Table 10).



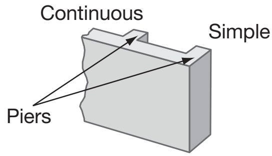

a Metal ties to columns. Simple support: direct force restraint limited to values given in Table 10



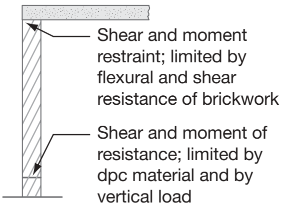

b Bonded return walls. Restrained support: direct force and moment restraint limited by tensile strength of masonry as given in Table 9



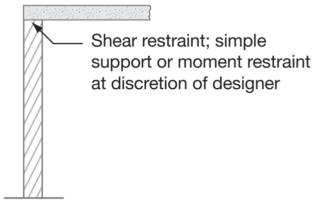

c Metal ties to columns or unbonded return walls. Shear and possibly moment restraint. Shear limited to values given in Table 10



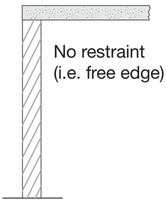

d Bonded to piers. As (b) for intermediate pier, as (a) for end pier



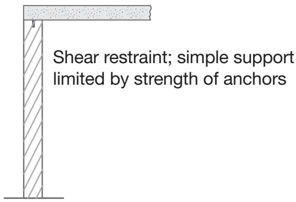

e In-situ floor slab cast onto wall span parallel to wall




f (1) Precast units spanning parallel to wall
(2) Walls solidly pinned up to structure above




g Wall built up to but not pinned to structure above




h Similar to (g) with suitable anchors

**Fig. 24 Continuous and simple edge conditions**

## Table 15 Bending moment coefficients in laterally loaded wall panels

Notes

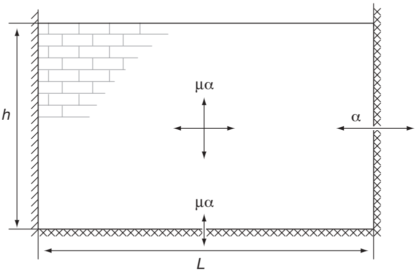
**a** Linear interpolation of μ and *h/L* is permitted.

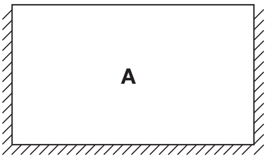
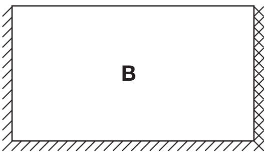
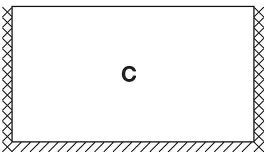
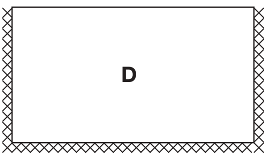
**b** When the dimensions of a wall are outside the range of *h/L* given in this Table, it will usually be sufficient to calculate the moments on the basis of a simple span. For example, a panel of type A having *h/L* greater than 1.75 will tend to span horizontally.

**c** See Fig. 24 for restraint conditions.

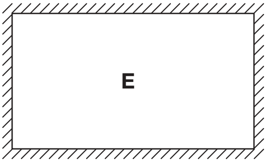
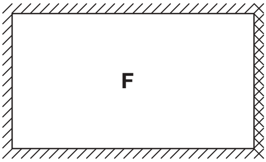
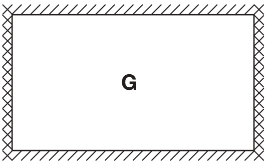
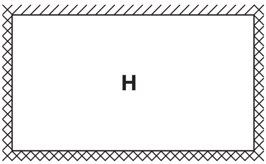
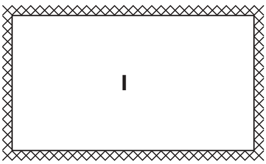
**Key to support conditions:**

- Denotes free edge
- Simply supported edge
- An edge over which full continuity exists

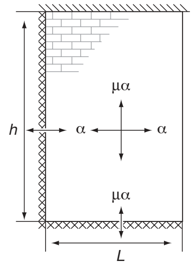



| Panel | μ | Values of α *h/L* 0.30 | 0.50 | 0.75 | 1.00 | 1.25 | 1.50 | 1.75 |
|---|---|---|---|---|---|---|---|---|
|  A | 1.00 | 0.031 | 0.045 | 0.059 | 0.071 | 0.079 | 0.085 | 0.090 |
| | 0.90 | 0.032 | 0.047 | 0.061 | 0.073 | 0.081 | 0.087 | 0.092 |
| | 0.80 | 0.034 | 0.049 | 0.064 | 0.075 | 0.083 | 0.089 | 0.093 |
| | 0.70 | 0.035 | 0.051 | 0.066 | 0.077 | 0.085 | 0.091 | 0.095 |
| | 0.60 | 0.038 | 0.053 | 0.069 | 0.080 | 0.088 | 0.093 | 0.097 |
| | 0.50 | 0.040 | 0.056 | 0.073 | 0.083 | 0.090 | 0.095 | 0.099 |
| | 0.40 | 0.043 | 0.061 | 0.077 | 0.087 | 0.093 | 0.098 | 0.101 |
| | 0.35 | 0.045 | 0.064 | 0.080 | 0.089 | 0.095 | 0.100 | 0.103 |
| | 0.30 | 0.048 | 0.067 | 0.082 | 0.091 | 0.097 | 0.101 | 0.104 |
|  B | 1.00 | 0.024 | 0.035 | 0.046 | 0.053 | 0.059 | 0.062 | 0.065 |
| | 0.90 | 0.025 | 0.036 | 0.047 | 0.055 | 0.060 | 0.063 | 0.066 |
| | 0.80 | 0.027 | 0.037 | 0.049 | 0.056 | 0.061 | 0.065 | 0.067 |
| | 0.70 | 0.028 | 0.039 | 0.051 | 0.058 | 0.062 | 0.066 | 0.068 |
| | 0.60 | 0.030 | 0.042 | 0.053 | 0.059 | 0.064 | 0.067 | 0.069 |
| | 0.50 | 0.031 | 0.044 | 0.055 | 0.061 | 0.066 | 0.069 | 0.071 |
| | 0.40 | 0.034 | 0.047 | 0.057 | 0.063 | 0.067 | 0.070 | 0.072 |
| | 0.35 | 0.035 | 0.049 | 0.059 | 0.065 | 0.068 | 0.071 | 0.073 |
| | 0.30 | 0.037 | 0.051 | 0.061 | 0.066 | 0.070 | 0.072 | 0.074 |
|  C | 1.00 | 0.020 | 0.028 | 0.037 | 0.042 | 0.045 | 0.048 | 0.050 |
| | 0.90 | 0.021 | 0.029 | 0.038 | 0.043 | 0.046 | 0.048 | 0.050 |
| | 0.80 | 0.022 | 0.031 | 0.039 | 0.043 | 0.047 | 0.049 | 0.051 |
| | 0.70 | 0.023 | 0.032 | 0.040 | 0.044 | 0.048 | 0.050 | 0.051 |
| | 0.60 | 0.024 | 0.034 | 0.041 | 0.046 | 0.049 | 0.051 | 0.052 |
| | 0.50 | 0.025 | 0.035 | 0.043 | 0.047 | 0.050 | 0.052 | 0.053 |
| | 0.40 | 0.027 | 0.038 | 0.044 | 0.048 | 0.051 | 0.053 | 0.054 |
| | 0.35 | 0.029 | 0.039 | 0.045 | 0.049 | 0.052 | 0.053 | 0.054 |
| | 0.30 | 0.030 | 0.040 | 0.046 | 0.050 | 0.052 | 0.054 | 0.055 |
|  D | 1.00 | 0.013 | 0.021 | 0.029 | 0.035 | 0.040 | 0.043 | 0.045 |
| | 0.90 | 0.014 | 0.022 | 0.031 | 0.036 | 0.040 | 0.043 | 0.046 |
| | 0.80 | 0.015 | 0.023 | 0.032 | 0.038 | 0.041 | 0.044 | 0.047 |
| | 0.70 | 0.016 | 0.025 | 0.033 | 0.039 | 0.043 | 0.045 | 0.047 |
| | 0.60 | 0.017 | 0.026 | 0.035 | 0.040 | 0.044 | 0.046 | 0.048 |
| | 0.50 | 0.018 | 0.028 | 0.037 | 0.042 | 0.045 | 0.048 | 0.050 |
| | 0.40 | 0.020 | 0.031 | 0.039 | 0.043 | 0.047 | 0.049 | 0.051 |
| | 0.35 | 0.022 | 0.032 | 0.040 | 0.044 | 0.048 | 0.050 | 0.051 |
| | 0.30 | 0.023 | 0.034 | 0.041 | 0.046 | 0.049 | 0.051 | 0.052 |

### Table 15 (Continued)

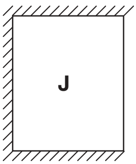
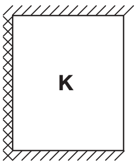
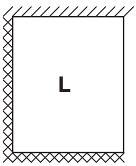
| | μ | Values of α | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | *h/L* | | | | | | |
| | | 0.30 | 0.50 | 0.75 | 1.00 | 1.25 | 1.50 | 1.75 |
|  E | 1.00 | 0.008 | 0.018 | 0.030 | 0.042 | 0.051 | 0.059 | 0.066 |
| | 0.90 | 0.009 | 0.019 | 0.032 | 0.044 | 0.054 | 0.062 | 0.068 |
| | 0.80 | 0.010 | 0.021 | 0.035 | 0.046 | 0.056 | 0.064 | 0.071 |
| | 0.70 | 0.011 | 0.023 | 0.037 | 0.049 | 0.059 | 0.067 | 0.073 |
| | 0.60 | 0.012 | 0.025 | 0.040 | 0.053 | 0.062 | 0.070 | 0.076 |
| | 0.50 | 0.014 | 0.028 | 0.044 | 0.057 | 0.066 | 0.074 | 0.080 |
| | 0.40 | 0.017 | 0.032 | 0.049 | 0.062 | 0.071 | 0.078 | 0.084 |
| | 0.35 | 0.018 | 0.035 | 0.052 | 0.064 | 0.074 | 0.081 | 0.086 |
| | 0.30 | 0.020 | 0.038 | 0.055 | 0.068 | 0.077 | 0.083 | 0.089 |
|  F | 1.00 | 0.008 | 0.016 | 0.026 | 0.034 | 0.041 | 0.046 | 0.051 |
| | 0.90 | 0.008 | 0.017 | 0.027 | 0.036 | 0.042 | 0.048 | 0.052 |
| | 0.80 | 0.009 | 0.018 | 0.029 | 0.037 | 0.044 | 0.049 | 0.054 |
| | 0.70 | 0.010 | 0.020 | 0.031 | 0.039 | 0.046 | 0.051 | 0.055 |
| | 0.60 | 0.011 | 0.022 | 0.033 | 0.042 | 0.048 | 0.053 | 0.057 |
| | 0.50 | 0.013 | 0.024 | 0.036 | 0.044 | 0.051 | 0.056 | 0.059 |
| | 0.40 | 0.015 | 0.027 | 0.039 | 0.048 | 0.054 | 0.058 | 0.062 |
| | 0.35 | 0.016 | 0.029 | 0.041 | 0.050 | 0.055 | 0.060 | 0.063 |
| | 0.30 | 0.018 | 0.031 | 0.044 | 0.052 | 0.057 | 0.062 | 0.065 |
|  G | 1.00 | 0.007 | 0.014 | 0.022 | 0.028 | 0.033 | 0.037 | 0.040 |
| | 0.90 | 0.008 | 0.015 | 0.023 | 0.029 | 0.034 | 0.038 | 0.041 |
| | 0.80 | 0.008 | 0.016 | 0.024 | 0.031 | 0.035 | 0.039 | 0.042 |
| | 0.70 | 0.009 | 0.017 | 0.026 | 0.032 | 0.037 | 0.040 | 0.043 |
| | 0.60 | 0.010 | 0.019 | 0.028 | 0.034 | 0.038 | 0.042 | 0.044 |
| | 0.50 | 0.011 | 0.021 | 0.030 | 0.036 | 0.040 | 0.043 | 0.046 |
| | 0.40 | 0.013 | 0.023 | 0.032 | 0.038 | 0.042 | 0.045 | 0.047 |
| | 0.35 | 0.014 | 0.025 | 0.033 | 0.039 | 0.043 | 0.046 | 0.048 |
| | 0.30 | 0.016 | 0.026 | 0.035 | 0.041 | 0.044 | 0.047 | 0.049 |
|  H | 1.00 | 0.005 | 0.011 | 0.018 | 0.024 | 0.029 | 0.033 | 0.036 |
| | 0.90 | 0.006 | 0.012 | 0.019 | 0.025 | 0.030 | 0.034 | 0.037 |
| | 0.80 | 0.006 | 0.013 | 0.020 | 0.027 | 0.032 | 0.035 | 0.038 |
| | 0.70 | 0.007 | 0.014 | 0.022 | 0.028 | 0.033 | 0.037 | 0.040 |
| | 0.60 | 0.008 | 0.015 | 0.024 | 0.030 | 0.035 | 0.038 | 0.041 |
| | 0.50 | 0.009 | 0.017 | 0.025 | 0.032 | 0.036 | 0.040 | 0.043 |
| | 0.40 | 0.010 | 0.019 | 0.028 | 0.034 | 0.039 | 0.042 | 0.045 |
| | 0.35 | 0.011 | 0.021 | 0.029 | 0.036 | 0.040 | 0.043 | 0.046 |
| | 0.30 | 0.013 | 0.022 | 0.031 | 0.037 | 0.041 | 0.044 | 0.047 |
|  I | 1.00 | 0.004 | 0.009 | 0.015 | 0.021 | 0.026 | 0.030 | 0.033 |
| | 0.90 | 0.004 | 0.010 | 0.016 | 0.022 | 0.027 | 0.031 | 0.034 |
| | 0.80 | 0.005 | 0.010 | 0.017 | 0.023 | 0.028 | 0.032 | 0.035 |
| | 0.70 | 0.005 | 0.011 | 0.019 | 0.025 | 0.030 | 0.033 | 0.037 |
| | 0.60 | 0.006 | 0.013 | 0.020 | 0.026 | 0.031 | 0.035 | 0.038 |
| | 0.50 | 0.007 | 0.014 | 0.022 | 0.028 | 0.033 | 0.037 | 0.040 |
| | 0.40 | 0.008 | 0.016 | 0.024 | 0.031 | 0.035 | 0.039 | 0.042 |
| | 0.35 | 0.009 | 0.017 | 0.026 | 0.032 | 0.037 | 0.040 | 0.043 |
| | 0.30 | 0.010 | 0.019 | 0.028 | 0.034 | 0.038 | 0.042 | 0.044 |

## Table 15 (Continued)




**Key to support conditions:**

- Denotes free edge
- Simply supported edge
- An edge over which full continuity exists

| | μ | Values of α | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | *h/L* | | | | | | |
| | | 0.30 | 0.50 | 0.75 | 1.00 | 1.25 | 1.50 | 1.75 |
|   | 1.00 | 0.009 | 0.023 | 0.046 | 0.071 | 0.096 | 0.122 | 0.151 |
| | 0.90 | 0.010 | 0.026 | 0.050 | 0.076 | 0.103 | 0.131 | 0.162 |
| | 0.80 | 0.012 | 0.028 | 0.054 | 0.083 | 0.111 | 0.142 | 0.175 |
| | 0.70 | 0.013 | 0.032 | 0.060 | 0.091 | 0.121 | 0.156 | 0.191 |
| | 0.60 | 0.015 | 0.036 | 0.067 | 0.100 | 0.135 | 0.173 | 0.211 |
| | 0.50 | 0.018 | 0.042 | 0.077 | 0.113 | 0.153 | 0.195 | 0.237 |
| | 0.40 | 0.021 | 0.050 | 0.090 | 0.131 | 0.177 | 0.225 | 0.272 |
| | 0.35 | 0.024 | 0.055 | 0.098 | 0.144 | 0.194 | 0.244 | 0.296 |
| | 0.30 | 0.027 | 0.062 | 0.108 | 0.160 | 0.214 | 0.269 | 0.325 |
|   | 1.00 | 0.009 | 0.021 | 0.038 | 0.056 | 0.074 | 0.091 | 0.108 |
| | 0.90 | 0.010 | 0.023 | 0.041 | 0.060 | 0.079 | 0.097 | 0.113 |
| | 0.80 | 0.011 | 0.025 | 0.045 | 0.065 | 0.084 | 0.103 | 0.120 |
| | 0.70 | 0.012 | 0.028 | 0.049 | 0.070 | 0.091 | 0.110 | 0.128 |
| | 0.60 | 0.014 | 0.031 | 0.054 | 0.077 | 0.099 | 0.119 | 0.138 |
| | 0.50 | 0.016 | 0.035 | 0.061 | 0.085 | 0.109 | 0.130 | 0.149 |
| | 0.40 | 0.019 | 0.041 | 0.069 | 0.097 | 0.121 | 0.144 | 0.164 |
| | 0.35 | 0.021 | 0.045 | 0.075 | 0.104 | 0.129 | 0.152 | 0.173 |
| | 0.30 | 0.024 | 0.050 | 0.082 | 0.112 | 0.139 | 0.162 | 0.183 |
|   | 1.00 | 0.006 | 0.015 | 0.029 | 0.044 | 0.059 | 0.073 | 0.088 |
| | 0.90 | 0.007 | 0.017 | 0.032 | 0.047 | 0.063 | 0.078 | 0.093 |
| | 0.80 | 0.008 | 0.018 | 0.034 | 0.051 | 0.067 | 0.084 | 0.099 |
| | 0.70 | 0.009 | 0.021 | 0.038 | 0.056 | 0.073 | 0.090 | 0.106 |
| | 0.60 | 0.010 | 0.023 | 0.042 | 0.061 | 0.080 | 0.098 | 0.115 |
| | 0.50 | 0.012 | 0.027 | 0.048 | 0.068 | 0.089 | 0.108 | 0.126 |
| | 0.40 | 0.014 | 0.032 | 0.055 | 0.078 | 0.100 | 0.121 | 0.139 |
| | 0.35 | 0.016 | 0.035 | 0.060 | 0.084 | 0.108 | 0.129 | 0.148 |
| | 0.30 | 0.018 | 0.039 | 0.066 | 0.092 | 0.116 | 0.138 | 0.158 |

### 5.4.3 Two-way spanning walls

BS5628-1[1] notes that the bending moments may be derived by yield-line theory. For panels without openings, the bending moments are:

$\alpha W_k \gamma_f L^2$, when the plane of failure is perpendicular to the bed joints, and
$\mu \alpha W_k \gamma_f L^2$, when the plane of failure is parallel to the bed joints

where

- $\gamma_f$ is the partial safety factor for loads from Table 8
- $\alpha$ is the bending moment coefficient taken from Table 15
- $W_k$ is the characteristic wind load per unit area
- $L$ is the length of the panel between supports
- $\mu$ 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 bricks in designation (iii) mortar: $\mu = 0.3/0.9 = 1/3$).
  Vertical loading increases the flexural strength of a panel in the parallel direction, in which case $\mu$ may be modified by using a flexural strength in the parallel direction of $f_{kx} + \gamma_m g_d$
  where
  - $\gamma_m$ is the appropriate partial safety factor for materials
  - $g_d$ is the design vertical dead load per unit area

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 subpanels, which can then be calculated using the rules given above (see Fig. 25). 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 can then be used instead of the moments obtained from the coefficients given in Table 15.

Very small openings in panels will have little effect on the strength of the panel in which they occur, and they normally need not be taken into account. Some guidance is given in clause 5.2.3.2.1 of BS5628-3[2].

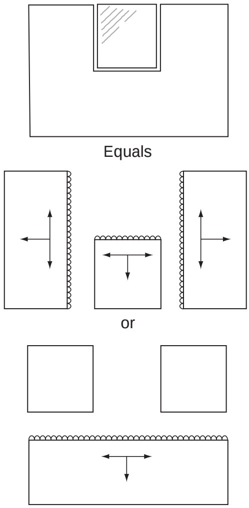



**Fig. 25 Example of subdivision of panel with openings**

### 5.4.4 One-way spanning walls

Walls spanning vertically may be treated as simply supported at the top and 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 the circumstances no additional tensile strength should be taken into account at this level.

The resistance moment at the base, generated by gravity action, is termed the stability moment of resistance, $MR_s$. If $MR_s$ exceeds $W_k\gamma_f h^2/8$, that of a propped cantilever, the wall should be designed as a propped cantilever (see Fig. 26).

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

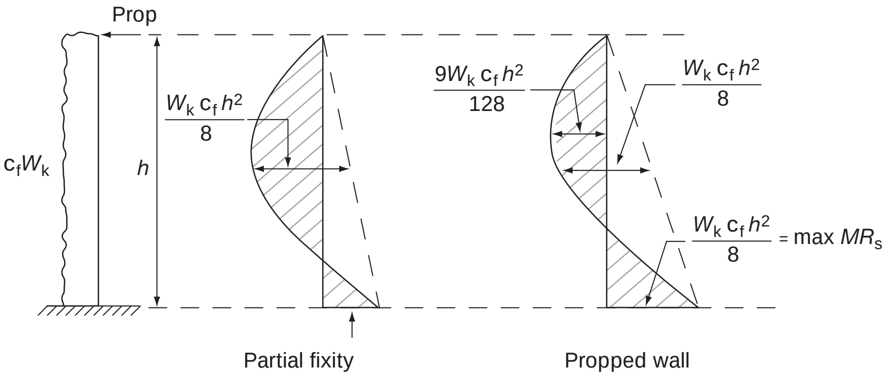



**Fig. 26 Moment diagrams for vertically spanning walls**

Where

| | |
|---|---|
| $W_k$ | is the characteristic wind load per unit area |
| $\gamma_f$ | is the partial safety factor for loads |
| $h$ | is the height of the panel |

but

| | |
|---|---|
| $h \leqslant 40t_{ef}$ | for simple support condition |
| $h < 45t_{ef}$ | is the characteristic wind load per unit area |
| $h \leqslant 50t_{ef}$ | is the partial safety factor for loads |

where $t_{ef}$ is the effective thickness of the wall panel.

The stability moment of resistance at base of wall is shown in Fig. 27 where

| | |
|---|---|
| $G_k$ | is the characteristic dead load |
| $\gamma_f$ | is the partial safety factor for loads |
| $f_k$ | is the characteristic compressive strength of masonry |
| $\gamma_m$ | is the appropriate partial safety factor for materials |
| $l_a$ | is the lever arm |
| $W_s$ | is the width of stressed area |

Stability moment of resistance at base is given by

$$MR_s = G_k \gamma_f l_a$$

The minimum width of wall ($W_s$) is stressed to ultimate ($1.1f_k/\gamma_m$, as derived in BS5628-1[1] Appendix B) to create the maximum lever arm about which the dead load of the wall rotates to generate the maximum stability moment of resistance.

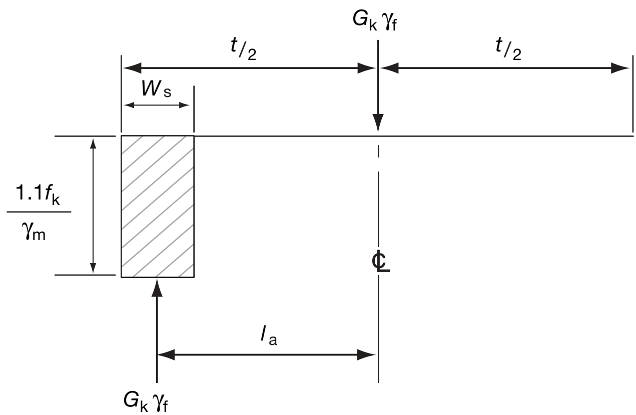



**Fig. 27 Stability moment of resistance at base of wall**

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

$$M_d = \left(\frac{f_{kx1}}{\gamma_m} + g_d\right) Z$$

where

| | |
|---|---|
| $f_{kx1}$ | is the characteristic flexural strength of masonry bending about an axis parallel to bed joints |
| $\gamma_m$ | is the appropriate partial safety factor for materials |
| $g_d$ | 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. |

In calculating $G_k$ and $g_d$ 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 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 Fig. 26 relates to a single-storey structure or the top storey in a multi-storey building.

The design analysis relating to Fig. 26 is at variance with clause 36.9.1 of BS5628-1[1] but is considered to provide a practical solution. Clause 36.9.1 of BS5628-1 does not specifically restrict the applied base moment to that of a propped cantilever but it does impose an additional ‘cracked section’ check for stability in the height (upper level) of the wall. This additional check is based upon the variation of flexural strengths between clay brickwork and concrete blockwork.

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), use $f_{kx2}$ (bending perpendicular to the bed joints); always take $g_d$ 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.

BS5628-1 suggests, in clause 36.4.4, the development of lateral load resistance through the in-plane arching action of a wall panel. Shrinkage effects in concrete blockwork and potentially inadequate frame support details, etc. can make this method of analysis unreliable, and it is not, in general, recommended.

### 5.4.5 Cavity walls

Provided that the wall ties used are capable of transmitting the compressive 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.

### 5.4.6 Geometric walls

The design procedure for geometric walls subject to lateral loading, such as diaphragm and fin walls (see Figs 28 and 29), is essentially the same at that given in subsection 5.4.4 for vertically

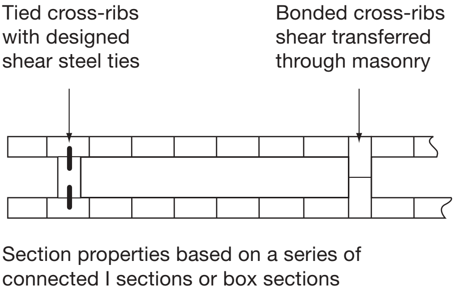



**Fig. 28 Typical diaphragm wall section**

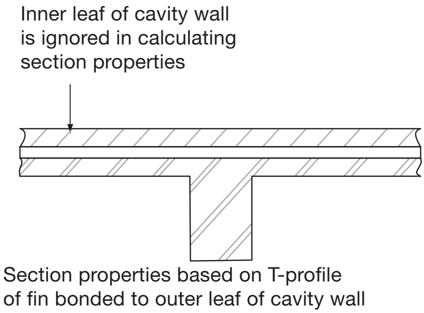



**Fig. 29 Typical fin wall section**

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.

Information on the design procedure for geometric walls subject to compressive loading can be found in references 38 and 39.

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 4 times the thickness of the flange when the flange is unrestrained or 6 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. Further information on geometric wall design may be obtained from reference 39.

A suggested design procedure applicable to both diaphragm and fin walls is:

(a) Calculate loadings (dead, imposed and wind).
(b) Select trial section of wall profile and masonry strength; suggested trial section selection procedures for both diaphragm and fin walls are given in subsection 5.4.7.
(c) Calculate applied bending moment at base of wall and compare with stability moment of resistance $MR_s$.
(d) Calculate position and magnitude of maximum applied bending moment within height of wall and compare with flexural resistance of wall at this level.
(e) Calculate shear stresses at junctions of cross-ribs (in diaphragm walls) and fin/flange (in fin walls); guidance is given in subsection 5.4.8 in respect of the shear calculation for a diaphragm wall.
(f) Design shear ties or calculate shear resistance of the bonded masonry at these shear interfaces.

Note: Because of the nature of differential movements between clay bricks and concrete blocks/bricks the mixing of these units within geometric wall profiles should be considered with caution.

The design procedure is one of trial and error and guidance is given in subsection 5.4.7 on the selection of trial sections for full analysis.

### 5.4.7 Trial sections

The symmetrical profile of a diaphragm wall permits the development of a direct route to a trial section which considers the two critical conditions that exist in the 'propped cantilever' action of the analysis.

Condition (i) exists at the base of the wall where the applied bending moment at this level must not exceed the stability moment of resistance of the wall.

Condition (ii) exists at approximately $3h/8$ down from the top of the wall where the flexural tensile stresses are a maximum and must not exceed those allowable through calculation.

Two graphs have been plotted (Figs 30 and 31) relating to these two conditions and for various values of wind loading $W_k$.

Then, for a known wall height and wind pressure, values of $K_2$ and $Z$ may be read off Figs 30 and 31 and, using Table 16, the most suitable section can be obtained for full analysis. It should be remembered that the two trial section graphs have been drawn assuming fixed conditions for a number of variable quantities, namely:

(a) wall acts as a true propped cantilever
(b) dpc at base of wall cannot transfer tension
(c) vertical roof loads (downward or uplift) are ignored
(d) $\gamma_m$ is taken to be 2.5
(e) $f_{kx}$ is taken to be 0.4N/mm$^2$
(f) density of masonry is taken to be 20kN/m$^3$
(g) $K_2$ values calculated using approximated lever arm method.

Note: The trial section graphs opposite are based on the loading combination of dead plus wind for which the partial safety factors on loads ($\gamma$f) are taken as 0.9 and 1.4 respectively.

Figs 30 and 31 should be used only for the purpose of obtaining a trial section, and a full analysis of the selected section should always be carried out.

Because of the asymmetrical shape of the fin wall it is not possible to derive a direct route to a trial section; however a trial section can be reasonably obtained by providing a section that has a stability moment of resistance $MR_s$, at the level of $MB$, equal to $W_{k1}\gamma_f\, h^2/8$ under wind pressure loading $W_{k1}$, i.e. when rotation at the base of the wall is about the face of the flange. For the purpose of the trial section assessment, the stability moment of resistance can be simplified to $\Omega H$, where $\Omega$ = trial section coefficient from Table 17.

### 5.4.8 Shear analysis

The shear stress at the cross-rib/flange interfaces of a diaphragm wall is calculated as illustrated in Fig. 32. A similar analysis method may be developed for other geometric profiles.

Vertical design shear stress,

$$v = \frac{VA\bar{y}}{Ib}$$

where

$V$ is the design shear force = $\gamma_f$ x characteristic shear force
$A$ is the effective flange area = $B$ x t1
$b$ = $t_w$
$I$ is the second moment of area of effective section
$\bar{y}$ is the distance from the neutral axis to the centroid of the effective flange.

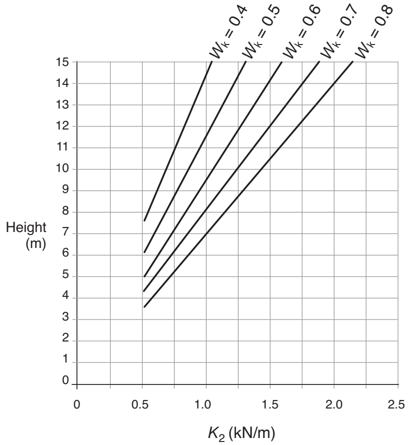



**Fig. 30 Diaphragm wall trial section: condition (i)**

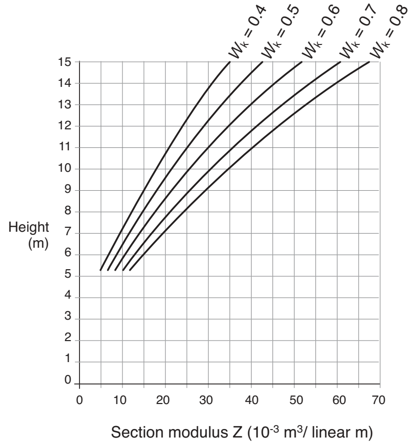



**Fig. 31 Diaphragm wall trial section: condition (ii)**

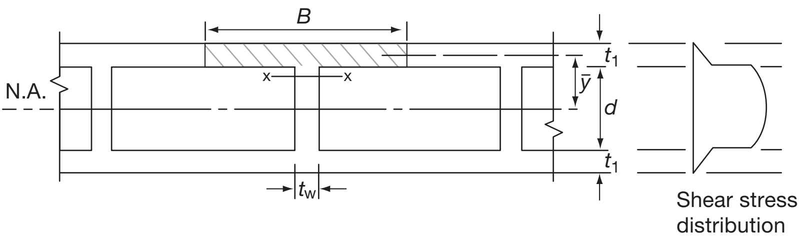



**Fig. 32 Diaphragm wall - shear analysis**

The vertical shear resistance at the interface of the cross-rib and flange is provided either by the bonded masonry for which characteristic shear strengths are given in subsection 4.3.3 or by metal shear connectors, the size and spacing of which should be calculated:

$$ru = \frac{12 t_w S_v}{0.87\ f_y}$$

where

$r$ is the width of connector
$u$ is the thickness of connector
$t_w$ is the width of masonry section in vertical shear
$S$ is the vertical spacing of connectors
$v$ is the design vertical shear stress on masonry section (as calculated above)
$f_y$ is the characteristic tensile strength of connector

**Table 16 Diaphragm wall – section properties**

| Section | Dimensions (in metres) | | | | Section properties/ diaphragm | | | Section properties/metre | | | Stability moment coefficient $K^2$ (kN/m) density = 20kN/m$^3$ | $K_2$ when density = 18kN/m$^3$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | $D$ | $d$ | $B$ | $b$ | $I$x10$^{-3}$ (m$^4$) | $Z$x10$^{-3}$ (m$^3$) | A (m$^2$) | $I$x10$^{-4}$ (m$^2$) | $Z$x10$^{-3}$ (m$^3$) | A (m$^2$) | | |
| 1 | 0.44 | 0.235 | 1.4625 | 1.36 | 8.91 | 40.49 | 0.324 | 6.09 | 27.69 | 0.222 | 0.835 | 0.752 |
| 2 | 0.44 | 0.235 | 1.2375 | 1.135 | 7.55 | 34.32 | 0.278 | 6.10 | 27.73 | 0.225 | 0.846 | 0.762 |
| 3 | 0.44 | 0.235 | 1.0125 | 0.91 | 6.21 | 28.83 | 0.232 | 6.13 | 27.88 | 0.229 | 0.862 | 0.776 |
| 4 | 0.5575 | 0.352 | 1.4625 | 1.36 | 16.18 | 58.04 | 0.337 | 11.06 | 39.69 | 0.230 | 1.097 | 0.987 |
| 5 | 0.5575 | 0.352 | 1.2375 | 1.135 | 13.74 | 49.29 | 0.290 | 11.10 | 39.83 | 0.234 | 1.116 | 1.004 |
| 6 | 0.5575 | 0.352 | 1.0125 | 0.91 | 11.31 | 40.57 | 0.244 | 11.17 | 40.07 | 0.241 | 1.149 | 1.034 |
| 7 | 0.665 | 0.46 | 1.4625 | 1.36 | 24.81 | 74.62 | 0.347 | 16.96 | 51.02 | 0.237 | 1.348 | 1.212 |
| 8 | 0.665 | 0.46 | 1.2375 | 1.135 | 21.12 | 63.52 | 0.301 | 17.07 | 51.33 | 0.243 | 1.382 | 1.243 |
| 9 | 0.665 | 0.46 | 1.0125 | 0.91 | 17.43 | 52.43 | 0.254 | 17.21 | 51.77 | 0.251 | 1.427 | 1.284 |
| 10 | 0.7825 | 0.5775 | 1.4625 | 1.36 | 36.56 | 93.45 | 0.359 | 24.99 | 63.90 | 0.245 | 1.639 | 1.478 |
| 11 | 0.7825 | 0.5775 | 1.2375 | 1.135 | 31.18 | 79.69 | 0.313 | 25.19 | 64.40 | 0.253 | 1.693 | 1.523 |
| 12 | 0.7825 | 0.5775 | 1.0125 | 0.91 | 25.82 | 66.01 | 0.267 | 25.50 | 65.20 | 0.264 | 1.766 | 1.590 |
| 13 | 0.89 | 0.685 | 1.4625 | 1.36 | 49.46 | 111.14 | 0.37 | 33.82 | 76.00 | 0.253 | 1.994 | 1.733 |
| 14 | 0.89 | 0.685 | 1.2375 | 1.135 | 42.4 | 95.3 | 0.324 | 34.26 | 77.01 | 0.262 | 1.994 | 1.794 |
| 15 | 0.89 | 0.685 | 1.0125 | 0.91 | 34.86 | 78.34 | 0.278 | 34.43 | 77.37 | 0.274 | 2.085 | 1.877 |

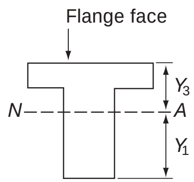
Note

For Sections 1, 4, 7, 10 and 13 the flange length slightly exceeds the limitations given in clause 36.4.3(b) BS5628-1[1]. These sections have been included since they are the closest brick sizes to the flanges recommended in the code. If the designer is concerned at this marginal variation calculation of the section properties may be made on the basis of an effective flange width of 1.33m.

| **Table 17 Fin wall – section properties – references A – H** | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Fin reference | A | B | C | D | E | F | G | H |
| Fin size (mm) | 665 x 327 | 665 x 440 | 778 x 327 | 778 x 440 | 890 x 327 | 890 x 440 | 1003 x 327 | 1003 x 440 |
| Effective width of flange (m) | 1.971 | 3.084 | 1.971 | 2.084 | 1.971 | 2.084 | 1.971 | 2.084 |
| Neutral axis $Y_1$ (m) | 0.455 | 0.435 | 0.522 | 0.500 | 0.589 | 0.563 | 0.654 | 0.626 |
| Neutral axis $Y_2$ (m) | 0.210 | 0.230 | 0.256 | 0.278 | 0.301 | 0.327 | 0.349 | 0.377 |
| Effective area (m$^2$) | 0.386 | 0.4611 | 0.4262 | 0.5152 | 0.4595 | 0.5601 | 0.4965 | 0.6098 |
| o.w. of effective area per m height $W$ (kN) | 7.720 | 9.222 | 8.458 | 10.216 | 9.190 | 11.202 | 9.930 | 12.196 |
| $I_{NA}$ (m$^4$) | 0.01567 | 0.01939 | 0.02454 | 0.0303 | 0.0359 | 0.04426 | 0.05021 | 0.06187 |
| $Z_1$ (m$^3$) | 0.03441 | 0.0445 | 0.04684 | 0.06059 | 0.06096 | 0.07862 | 0.07677 | 0.09883 |
| $Z_2$ (m$^3$) | 0.07462 | 0.0843 | 0.09663 | 0.10898 | 0.11928 | 0.13536 | 0.14387 | 0.16410 |
| Trial section coefficient Ω (kNm/m) | 1.6212 | 2.1210 | 2.1483 | 2.840 | 2.7662 | 3.6631 | 3.4656 | 4.5978 |

| **Table 17 (Continued) Fin wall – section properties – references J – R** | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Fin reference | J | K | L | M | N | P | Q | R |
| Fin size (mm) | 1115 x 327 | 1115 x 440 | 1227 x 327 | 1227 x 440 | 1339 x 327 | 1339 x 440 | 1451 x 327 | 1451 x 440 |
| Effective width of flange (m) | 1.971 | 3.084 | 1.971 | 2.084 | 1.971 | 2.084 | 1.971 | 2.084 |
| Neutral axis $Y_1$ (m) | 0.718 | 0.687 | 0.780 | 0.747 | 0.841 | 0.807 | 0.902 | 0.866 |
| Neutral axis $Y_2$ (m) | 0.397 | 0.428 | 0.447 | 0.480 | 0.498 | 0.532 | 0.549 | 0.585 |
| Effective area (m$^2$) | 0.5331 | 0.6591 | 0.5697 | 0.7084 | 0.6064 | 0.7577 | 0.6430 | 0.807 |
| o.w. of effective area per m height $W$ (kN) | 10.662 | 13.182 | 11.394 | 14.168 | 12.128 | 15.154 | 12.860 | 16.140 |
| $I_{NA}$ (m$^4$) | 0.06746 | 0.08312 | 0.088 | 0.10848 | 0.11208 | 0.13826 | 0.13992 | 0.17277 |
| $Z_1$ (m$^3$) | 0.09395 | 0.12099 | 0.11282 | 0.14522 | 0.13327 | 0.17132 | 0.15513 | 0.1995 |
| $Z_2$ (m$^3$) | 0.16992 | 0.19421 | 0.19687 | 0.226 | 0.22506 | 0.26039 | 0.25487 | 0.2953 |
| Trial section coefficient Ω (kNm/m) | 4.2328 | 5.6419 | 5.0931 | 6.8006 | 6.0397 | 8.0619 | 7.0601 | 9.4419 |

Trial section coefficient $\Omega = W\,Y_2$



### 5.4.9 Design loading cases

The diaphragm wall, being a symmetrical profile, is relatively straightforward to analyse. The fin wall, being asymmetrical, requires more careful analysis for the 2-directional loading cases of wind pressure and wind suction. Both loading directions require analysis at base level and within the height of the fin wall. The centres of the cross-ribs or of the fins may be dictated by the capacity of their flanges to span horizontally between them and this is an essential check in the design process.

# 6 Details and construction

## 6.1 Sequence of building and how it affects design

As with all structures, the construction details should be considered at the design stage. Most loadbearing masonry buildings rely for stability on items installed later, such as floors and roofs. The design should avoid (if possible) excessively slender or unbuttressed members that, although stable in the finished building, may require temporary propping during construction.

## 6.2 Types of walls

### 6.2.1 Solid walls

Half-brick or block walls are usually built in stretcher bond, and full-brick walls in a range of bonds that include a mixture of headers and stretchers (e.g. English or Flemish bonds) to give proper bonding throughout both the length and thickness of the wall.

Where solid walls are to be built of two skins in stretcher bond and it is intended that the two skins act as a composite wall, metal ties or reinforcement across the 'collar' joint between the two skins are required to connect them. The collar joint should not exceed 25mm and be solidly filled with mortar as the work proceeds. Certain requirements, including a minimum thickness for each leaf are given in BS5628-3[2] clause 29.6. In the case of a collared wall in blockwork its compressive strength should be reduced from $f_k$ to $0.9f_k$

### 6.2.2 Cavity walls

Each leaf of a cavity wall may be of brick or of block and should not be less than 75mm thick. The width of cavity will normally be not less than 50mm, but where wider cavities are required (e.g. to accommodate cavity insulation), the cavity width may be up to 300mm. Where either of the leaves is less than 90mm thick the cavity should not be wider than 75mm.

The two leaves of the cavity wall should be tied together by metal ties. The selection of wall ties is given in Table 6 and their recommended spacing in Table 18. At the sides of openings and at free ends, vertical spacing of the ties should be reduced to 300mm and be placed within 225mm of the end of the wall or opening (see Fig. 33). The minimum embedment of the ties in the mortar should be 50mm in each leaf.

## 6.3 Lateral restraints, straps and ties

Details at points of horizontal or vertical lateral restraint should be examined so that requisite support and strength is achieved.

Section 3.6 describes the requirements for restraining straps or ties and holding-down straps.

**Table 18 Wall ties according to BS5628-3[2] [(a)]**

(A) Spacing of ties

| Least leaf thickness (one or both) (mm) | Type of tie | Cavity width (mm) | Equivalent no. of ties per square metre | Horizontally (mm) | Vertically (mm) |
|---|---|---|---|---|---|
| 65 to 90 | All | 50 to 75 | 4.9 | 450 | 450 |
| >90 | See (B) | 50 to 300 | 2.5 | 900 | 450 |

(B) Selection of ties: types and lengths

| Least leaf thickness (one or both) (mm) | Normal cavity width[c] (mm) | Permissible type of tie[b] | | |
|---|---|---|---|---|
| | | Tie length[d] | Shape name in accordance with BS1243[25] | Type number in accordance with DD140-2[26] |
| 75 | not greater than 75 | 200 | Butterfly, double triangle or vertical twist | Types 1, 2, 3 or 4 selected on the basis of the design loading and design cavity width. Prescriptive rules for selection, and a model calculation, are given in DD140-2 |
| 90 | 76 to 90 | 225 | Double triangle[e] or vertical twist | |
| 90 | 91 to 100 | 225 | Double triangle[f] or vertical twist | |
| 90 | 101 to 125 | 250 | Vertical twist | |
| 90 | 126 to 150 | 275 | Vertical twist | |
| 90 | 151 to 175 | 300 | Vertical twist | |
| 90 | 176 to 300 | d | Vertical twist style[g] | |

Notes

**a** The Building Regulations[33] section A1/2 suggests a horizontal spacing of 750mm for a cavity width of 76 to 100mm. It should be noted that section A1/2 applies to residential buildings, small single storey non residential buildings and small annexes to residential buildings and is based on low strength bricks and blocks.

**b** Or equivalent safety tie

**c** Where face insulated blocks are used the cavity width should be measured from the face of the concrete.

**d** Tie lengths are given, in 25mm increments, that best meet the performance recommendation that embedment depths are not less than 50mm in both leaves, taking into account building and construction tolerances, and that ties must not protrude from the face. For cavities wider than 180mm the length should be calculated as the structural cavity width plus 125mm and the nearest stock length selected, reference should also be made to BS5628-3.

**e** The minimum length exceeds the maximum length specified under BS1243, but 225mm double triangle ties, which otherwise conform to BS1243 should be suitable.

**f** Reference to be made to BS5628-3 and specialist tie manufacturers.

**g** The strength and stiffness of masonry/masonry ties to DD140 ranges from type 1, the stiffest, to Type 4, the least stiff. For ties to BS1243 the vertical twist is the stiffest and the butterfly the least stiff.

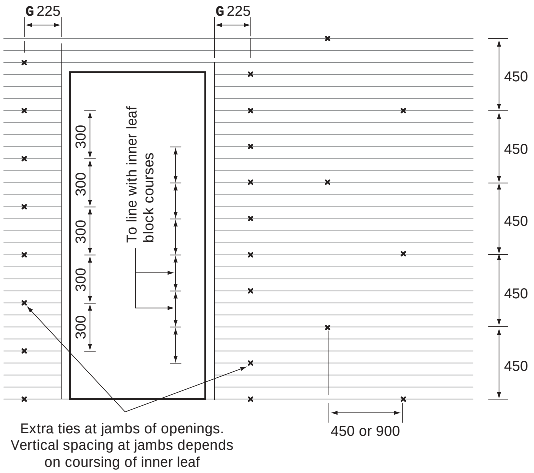



**Fig. 33 Spacing of ties at openings**

Note This figure is based on the recommendations of BS5628-3. The NHBC Standards[40] recommends the extra ties at jambs of openings to be within 150mm of the opening. All dimensions are in mm.

## 6.4 Mixing of framed construction and loadbearing masonry

This mixing falls broadly into three situations. The first and most commonly adopted form of construction is where the framed construction and the loadbearing masonry parts of the structure are structurally independent of each other. In this case, there is simply the need to provide adequate separation to accommodate relative movements between the two structural systems. These movements arise from thermal or moisture changes, loading causing sway or differential settlement.

The second situation arises where one form is built on top of the other, and the two constructions can be easily separated, e.g.:

- a penthouse flat may be built in loadbearing masonry on top of an office block in framed construction
- the upper floors of a block of flats may be built in loadbearing masonry on a ground floor of framed construction where other considerations dictate an open-plan solution (podium construction).

The third situation, not commonly in current use, is where the frame construction and the loadbearing masonry construction each provide essential structural support to the other. If such a solution is contemplated, it is important to consider the following points:

- structural stability during the construction
- the interdependence of the two systems; this should be noted on the drawings, especially where it is not completely obvious, and where there is the possibility of structural alterations in the future that might affect stability
- relative movements, from whatever cause, of the mixed systems
- two different trades working simultaneously may confuse the sequence of work; savings in material costs may therefore not result in overall savings in construction costs.

### 6.5 Overhangs, corbels, cornices: difficulties with suspending bricks from ironmongery

Such details are usually expensive to construct. Simple details are often better and cheaper. It is important to consider construction tolerances when formulating details, although simplicity in detailing should be the aim. Details using special ironmongery items that allow for large tolerance in all directions are usually expensive. Such ironmongery will usually have little cover/protection to weather, and to enhance durability should be of suitable austenitic stainless steel. It is important to consider the design effects of such details, e.g. are any moments applied to the masonry?

### 6.6 Bonding and coursing

For both vertical and lateral loading the structural integrity of the masonry depends on proper bonding; each masonry unit should overlap the one below by not less than one-quarter of its length. When the design is based on lesser bonding e.g. stack bonding, which introduces continuous vertical joints into the masonry, prefabricated bed-joint reinforcement should be introduced (austenitic stainless steel in external walls). In the case of a cavity wall with the outer leaf stack bonded it may be preferable to use adequate ties and take all load on the inner leaf. Bed-joint reinforcement can also be used to control cracking arising from the influence of openings, deflections, etc. (NB: reinforced masonry is not covered by this *Manual*).

The proper bonding of piers, buttresses and fins should also be considered if continuous perpend joints are to be avoided. Straps, ties or bed-joint reinforcement may be used in lieu of bonding where non-loadbearing walls abut loadbearing masonry.

Cut bricks or blocks in loadbearing construction are undesirable. The clear storey heights, between floors, of walls, piers or columns should be a multiple of the co-ordinated size of the bricks or blocks. Although other brick sizes can be supplied, most bricks have a co-ordinating height of 75mm; the co-ordinating heights of blocks are generally 225mm and 200mm (modular).

From the above it is evident that where vertical elements comprise both brick and block some consideration should be given to the clear and, if appropriate, the floor-to-floor storey heights. Mixed block heights are possible but create onsite problems of ordering and use.

Problems of matching and coursing also affect the vertical centres of wall ties in cavity walling and the bonding of intersecting walls. Structural elements at floor level (slabs and beams) should also be a multiple of co-ordinated sizes of the masonry units, especially where the building is clad with masonry that ‘runs’ past the horizontal structure.

Excessive thickness of bed-joints (to make up the differences in coursing) will reduce the loadbearing capacity of the masonry and the weathertightness of external masonry, and should therefore be avoided. Good practice would normally be observed if the mean thickness of any mortar joint is not greater than 10 ± 3mm.

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 bricks, 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 brickwork overturning on the bed-joint and so that the mortar applied to the vertical brick face does not fall away before the brick is placed.

### 6.7 Lintels and lintel bearings

Unless the masonry over an opening is supported by other means (e.g. concrete slabs or beams) lintels will be required to support the masonry and transfer the loads to, the supporting walls or piers. It is essential for both solid and cavity construction to support the full thickness of the masonry wall over the full width of the opening.

For cavity walls comprising different materials with different movement characteristics, including deflections from the applied loading, separate lintels may be desirable over the opening and under the different leaves.

Consideration should be given to the flexural stiffness of lintels to avoid problems with deflection or horizontal movement/rotations at the supports, particularly over large openings or under large applied loads. In the case of prestressed, precast plank lintels, and certain cold-formed steel lintels that work with composite action with the wall over, it is necessary to provide the requisite number of courses to achieve the required structural interaction. (NB: These lintels are unsuitable for use where new openings are formed in existing construction.)

The length of bearing required to transfer the load from the lintel to the supporting masonry should be assessed and specified but in any case should not be less than 100mm in length (150mm for pressed-steel lintels). In cases of highly loaded bearings, padstones may be required.

Certain types of cellular, frogged or hollow masonry units that are normally suitable for the construction of a wall may not provide sufficient bearing strength at points of concentrated load and may need to be filled. Similarly, in the case of in-situ concrete lintels, slabs and beams, capping the top course of masonry or filling the hollows of voided masonry units should be undertaken.

### 6.8 Filling of mortar joints

Except in the special case of shell bedding to wide hollow block masonry, the bed and perpend joints of brick or block walls should be completely filled with mortar. Incompletely filled joints should not be permitted because:

- the axial load capacity of wall is reduced
- flexural strength may be reduced
- ties, straps or anchors may be insufficiently embedded
- risk of water penetration increases, leading to potential durability problems.
- a fire hazard may be introduced
- resistance to the passage of sound will be reduced

To achieve complete filling of the bed-joints, single frogged bricks should be laid frog- up and double frogged bricks laid with larger frog-up. Cellular blocks, however, should be laid with their hollow cavity downwards. In the case of the perpend joints, ‘tipping and tailing’ of the vertical arisses of the masonry units should be discouraged.

Tooled mortar joints are more resistant to rain penetration than untooled joints (see subsection 3.3.2). Recessed mortar joints also increase the risk of rain penetration especially where the masonry units are perforated or hollow.

### 6.9 Damp-proof courses (dpcs)

Careful consideration is required so that the details of both the horizontal and vertical dpcs 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 dpcs whether horizontal or vertical, and the junctions between horizontal and vertical dpcs, may, if not properly designed or considered, direct water into the building. In the case of cavity trays, lack of support to the dpc over the cavity may also lead to water ingress (see BS 5628-3[2], section 5.5.5 for typical details).

Horizontal, flexible dpcs should be sandwiched in a mortar bed and two courses of brick or one course of block built immediately so that there is minimum disturbance of the mortar joint between the bricks and the dpc until the mortar has set. Clay brick dpcs should be constructed in the same way as normal brickwork masonry, not less than two courses high, with staggered cross-joints and laid in a designation (i) mortar.

Flexible dpcs 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 brick perpend joint in the external leaf should be left unfilled in the masonry course immediately above the dpc to allow any water retained by the dpc or cavity tray to drain out.

Dpcs, whether flexible or rigid, should not be pointed or rendered over since this will allow water to by-pass the dpc.

### 6.10 Chases and holes

Chases and holes, depending on their size and location, may represent a reduction in loadbearing capacity of the masonry and should be allowed for in the design. Horizontal chases reduce the capacity against lateral loading. They also reduce the bearing width and introduce eccentricities in vertical loads. Vertical chases lessen the capacity of a wall to resist horizontal loads and act as a stress inducer for shrinkage or expansion cracking.

Chasing should not be permitted in hollow or cellular block walls and should be carefully considered in walls constructed of perforated bricks or with recessed mortar joints.

Small circular holes that can be made by drilling or coring may be formed after the construction of the masonry wall but larger holes should preferably be square, of dimensions to suit the masonry units size and coursing, and formed at the time of construction of the wall.

Large holes, e.g. greater than $1^1/_2$ masonry units wide, may require a lintel or similar over the opening to facilitate construction.

Holes and chases formed after the construction should not be made by impact methods, as these can encourage local cracking that may propagate under loads and movements.

### 6.11 Movement joint details

After construction, buildings are subject to dimensional changes, which may be caused by one or more of the following factors:

- change in temperature
- seasonal change in moisture content
- long-term absorption of water vapour
- chemical action, e.g. carbonation
- deflection of supporting structure under loads/creep
- ground movement/differential settlement

In general, because restraints are often present, masonry is not completely free to move, and forces may develop that may lead to bowing or cracking. Masonry units of markedly different characteristics should not be bonded but should be effectively separated by a movement joint or slip plane. It is essential to consider provision for movement at the design stage. Annex B of BS5628-3[1] gives information on various movements that can occur in masonry and a method of predicting the degree of movement likely to occur.

Proper movement joints need, therefore, to be included at appropriate intervals to allow for thermal and other types of movement in the structure. Such movement will, of course, act in the vertical as well as horizontal direction.

Materials used in buildings have different rates of thermal and other types of movement. Fig. 34 gives approximate coefficients of thermal expansion per $^{o}$C change in temperature, and Table 19 indicates the range of moisture shrinkage for different materials.

Where different materials are connected together or connected to parts of a building not subject to external changes of temperature, care has to be taken in design to accommodate the expansion and contraction of one relative to another to limit and control cracking.

Joints accommodating horizontal movement in clay masonry walls may be spaced at up to 15m maximum centres. For practical purposes and to keep joint widths to acceptable dimensions, movement joints are more usually spaced at intervals of about 8 to 12m centres. For calcium silicate and sand-lime brickwork the joints for accommodating movements should be at intervals of between 7.5 and 9m maximum centres.

In concrete masonry, vertical joints to accommodate horizontal movement should be provided at intervals of about 6 to 8m, and it should be noted that the risk of cracking increases if the length of a panel exceeds twice the height. It is always desirable to consult the manufacturer before using joint spacing greater than 6m.

Movement joints to accommodate vertical movements or deflections cannot be incorporated in vertically loaded walls but may be necessary for laterally loaded infill panels. Such joints are usually at the underside of a floor or roof, and when required by the design need to incorporate means of lateral restraint to the panel.

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 joint filler and sealant may need to be considered.

Typical vertical movement joint widths in clay brickwork are about 15 to 20mm wide and those in calcium silicate masonry (where there is some permanent shrinkage) 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, and up to 20mm for south and west facing external walls.

In the case of horizontal movement joints the necessary joint width is 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 necessary if the masonry is not to be subjected to any imposed loads.

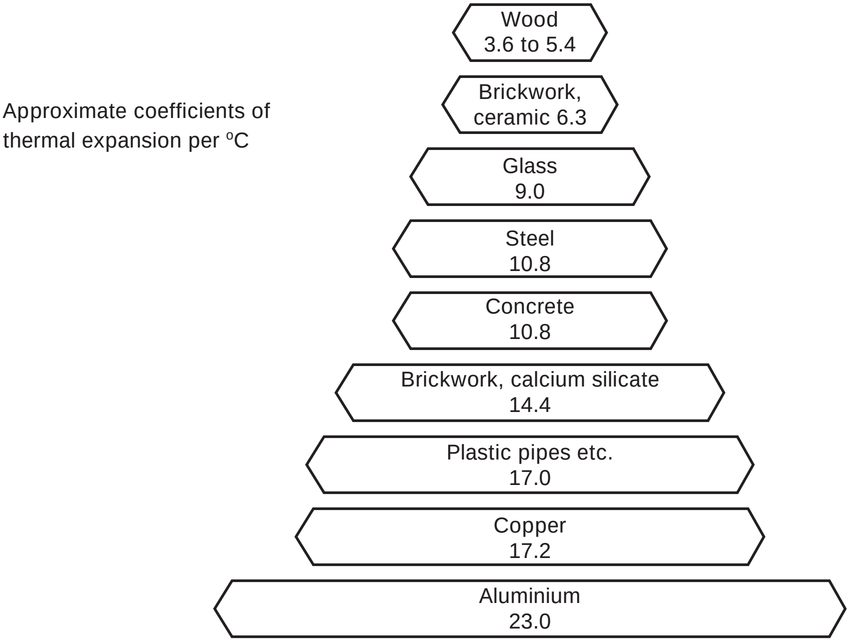



**Fig. 34 Coefficient of thermal expansion of different materials**

| **Table 19 Range of moisture shrinkage for different materials** | |
|---|---|
| Many constructional materials shrink on drying and expand again on wetting, this process being partially or wholly reversible. The following values show the order of magnitude of shrinkage when walls, etc, dry out in air at 65% RH. | |
| Material | Approximate shrinkage (%) |
| Metals | None |
| Brickwork ceramic, clay | Negligible |
| Brickwork, calcium silicate | 0.01 to 0.035 |
| Concrete | 0.03 to 0.12 |
| Wood | 2.0 to 4.0 (across the grain)<br>0.1 (along the grain) |
| On exposure to air all newly fired ceramic materials take up moisture, causing them to expand slightly. Clay bricks share this undesirable phenomenon, known as moisture expansion<br>The greater part of the expansion occurs within a few hours of the bricks leaving the kiln, therefore, the use of kiln fresh bricks should be avoided whenever possible. However it must be remembered that movement continues at a decreasing rate for many years. | |

## 6.12 Partition walls

Internal walls not carrying defined loadings from the frame or structure, including lateral loads, can be sized by reference to Fig. 35 (from Fig. 6, BS5628-3[1]).

## 6.13 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.

The permissible deviations given in BS5628-3[2] Table A2 should be taken as indicative only, as other requirements may necessitate a greater degree of accuracy being achieved.

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.

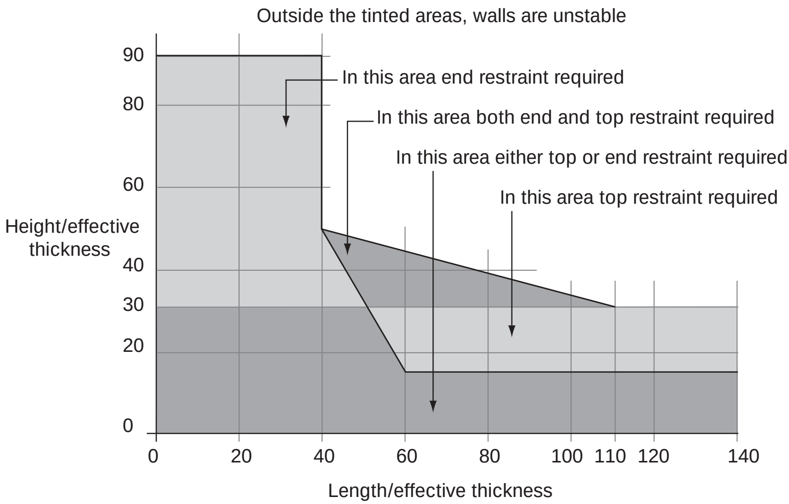



**Fig. 35 Limiting dimensions of walls for stability**

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 site-made 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.

Whenever relevant, any details and allowances for dimensional inaccuracies should include an assessment of the lack-of-fit of any necessary anchorages, lateral restraints, ties and straps.

### 6.14 Inspections and acceptance

The quality of the construction is the contractor's responsibility, and examinations 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. Table 20 indicates the probable reduction in ultimate compressive strength caused by some common workmanship deflects.

The reduction in strength from unfilled perpend joints is mainly of significance in laterally loaded masonry.

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. BS8000-3[22] provides further information on this topic.

**Table 20 Reduction in ultimate compressive strength caused by some common workmanship defects**

| | |
|---|---|
| Furrowed bed-joints | 25% |
| 16mm bed-joints | 25% |
| 12mm bow or out-of-plumb | 15% |

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