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2005

Code of practice for the use of masonry —

Part 1: Structural use of unreinforced masonry

ICS 91.080.30

British Standards

Committees responsible for this British Standard

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British Masonry Society
British Precast Concrete Federation
CERAM Research Ltd.
Concrete Block Association
Construction Federation
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Contents

Foreword

This part of BS 5628 has been prepared under the direction of Subcommittee B/525/6, Use of masonry. It supersedes BS 5628-1:1992, which will be withdrawn on 31 March 2006.

BS 5628 is published in the following parts:

— Part 1: *Structural use of unreinforced masonry;*

— Part 2: *Structural use of reinforced and prestressed masonry*;

— Part 3: *Materials and components, design and workmanship*.

This edition of BS 5628-1 is a revision of the standard necessary to accommodate the publication of BS EN 771, BS EN 845 and BS EN 998 (see Clause **2**), and the consequent withdrawal of the standards that they superseded.

Where materials, components and methods of design and construction are not covered by this or any other British Standard, this does not discourage their use. The designer may need to be satisfied by reference to the appropriate manufacturers' literature and test certificates, if any, issued by competent, independent authorities to ensure that the materials and methods to be employed provide a level of performance at least equal to that recommended in this standard.

As a code of practice, this part of BS 5628 takes the form of guidance and recommendations. It should not be quoted as though it were a specification and particular care should be taken to ensure that claims of compliance are not misleading.

This standard was drafted on the assumption that the execution of its recommendations is entrusted to appropriately qualified and competent people.

A British Standard does not purport to include all the necessary provisions of a contract. Users of British Standards are responsible for their correct application.

Compliance with a British Standard does not of itself confer immunity from legal obligations.

Summary of pages

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Section 1. General

1 Scope

This Part of BS 5628 gives recommendations for the structural design of unreinforced masonry units of bricks, blocks, manufactured stone, square dressed natural stone, and random rubble masonry.

NOTE The thickness of a wall determined from strength considerations may not always be sufficient to satisfy requirements for other properties of the wall such as resistance to fire, thermal insulation, sound insulation or resistance to damp penetration and reference should be made to BS 5628-3 or BS 5390, as appropriate.

It has been assumed in the drafting of this code that the design of masonry is entrusted to chartered structural or civil engineers or other appropriately qualified persons, for whose guidance it has been prepared, and that the execution of the work is carried out under the direction of appropriately qualified supervisors.

2 Normative references

The following referenced documents are indispensable for the application of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

BS 3892-1, *Pulverized-fuel ash — Specification for pulverized-fuel ash for use with Portland cement.*

BS 4027, *Specification for sulfate-resisting Portland cement.*

BS 4729, *Specification for dimensions of bricks of special shapes and sizes.*

BS 5224, *Specification for masonry cement.*

BS 5628, *Code of practice for the use of masonry — Part 2: Structural use of reinforced and prestressed masonry.*

BS 5628:2005, *Code of practice for the use of masonry — Part 3: Materials and components, design and workmanship.*

BS 6399-1, *Design loading for buildings — Part 1: Code of practice for dead and imposed loads.*

BS 6399-2, *Design loading for buildings — Part 2: Code of practice for wind loads.*

BS 6399-3, *Design loading for buildings — Part 3: Code of practice for imposed roof loads.*

BS 6699, *Specification for ground granulated blastfurnace slag for use with Portland cement.*

BS 7979, *Specification for limestone fines for use with Portland cement.*

BS 8000, *Workmanship on building sites — Part 3: Code of practice for masonry.*

BS 8002, *Code of practice for earth retaining structures.*

BS 8103, *Structural design of low-rise buildings — Part 1: Code of practice for stability, site investigation, foundations and ground floor slabs for housing.*

BS 8215:1998, *Code of practice for design and installation of damp-proof courses in masonry construction.*

BS EN 450-1, *Fly ash for concrete — Definition, specifications and conformity criteria.*

BS EN 197, *Cement — Part 1: Composition, specifications and conformity criteria for common cements.*

BS EN 771-1, *Specification for masonry units — Clay masonry units.*

BS EN 771-2, *Specification for masonry units — Calcium silicate masonry units.*

BS EN 771-3, *Specification for masonry units — Aggregate concrete masonry units.*

BS EN 771-4, *Specification for masonry units — Autoclaved aerated concrete masonry units.*

BS EN 771-5, *Specification for masonry units — Manufactured stone masonry units.*

BS EN 771-6, *Specification for masonry units — Natural stone masonry units.*

BS EN 772-1, *Methods of test for masonry units — Determination of compressive strength.*

BS EN 772-7, *Methods of test for masonry units —- Determination of water absorption of clay masonry damp proof course units by boiling in water.*

BS EN 845, *Specification for ancillary components for masonry — Part 1: Ties, tension straps, hangers and brackets.*

BS EN 934, *Admixtures for concrete, mortar and grout — Part 3: Admixtures for masonry mortar. Definitions, requirements, conformity, marking and labelling.*

BS EN 998, *Specification for mortar for masonry — Part 2: Masonry mortar.*

BS EN 1015, *Methods of test for mortar for masonry — Part 2: Bulk sampling of mortars and preparation of test mortars.*

BS EN 1015, *Methods of test for mortar for masonry — Part 11: Determination of flexural and compressive strength of hardened mortar.*

BS EN 1052, *Methods of test for masonry — Part 1: Determination of compressive strength.*

BS EN 1052, *Methods of test for masonry — Part 2: Determination of flexural strength.*

BS EN 1052, *Methods of test for masonry — Part 3: Determination of initial shear strength.*

BS EN 1052, *Methods of test for masonry — Part 4: Determination of shear strength including damp proof course.*

BS EN 12878, *Pigments for the colouring of building materials based on cement and/or lime. Specifications and methods of test.*

DD 86, *Damp proof courses — Part 1: Methods of test for flexural bond strength and short term shear strength.*

DD 86, *Damp proof courses — Part 2: Method of test for creep deformation.*

3 Definitions

For the purposes of this part of BS 5628 the following definitions apply.

3.1
actual dimension
<either> work size of the unit

<or, where applicable for solid walls> sum of the work size of the units together with the work size of the joints between them

3.2
building classes

3.2.1
building class 1

— houses not exceeding four storeys;

— agricultural buildings;

— buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance 1.5 times the building height.

3.2.2
building class 2A

— five-storey single occupancy houses;

— hotels not exceeding four storeys;

— flats, apartments and other residential buildings not exceeding four storeys;

— offices not exceeding four storeys;

— industrial buildings not exceeding three storeys;

— retaining premises not exceeding three storeys of less than 2 000 m^2 floor area in each storey;

— single-storey educational buildings;

— all buildings not exceeding two storeys to which members of the public are admitted and which contain floor areas not exceeding 2 000 m^2 at each storey.

3.2.3
building class 2B

— hotels, flats, apartments and other residential buildings greater than four storeys but not exceeding 15 storeys;

— educational buildings greater than one storey but not exceeding 15 storeys;

— retailing premises greater than three storeys but not exceeding 15 storeys;

— hospitals not exceeding three storeys;

— offices greater than four storeys but not exceeding 15 storeys;

— all buildings to which members of the public are admitted which contain floor areas exceeding 2 000 m^2 but less than 5 000 m^2 at each storey;

— car parking not exceeding six storeys.

3.2.4
building class 3

— all buildings defined above as class 2A and 2B that exceed the limits on area and/or number of storeys;

— grandstands accommodating more than 5 000 spectators;

— buildings containing hazardous substances and/or processes.

3.3
Category I masonry unit
unit with a declared compressive strength, with a probability of failure to reach it not exceeding 5 %

NOTE This probability may be determined via the mean or characteristic value.

3.4
Category II masonry unit
unit not intended to conform to the level of confidence of Category I units

3.5
characteristic load
<ideally, where the load acts unfavourably> load which has a probability of not more than 5 % of being exceeded;

or,

<where the load acts favourably> load which has a probability of at least 95 % of being exceeded

NOTE In practice, the value of a characteristic load is obtained from the appropriate British Standard.

3.6
characteristic strength of masonry
value of the strength of masonry below which the probability of test results falling is not more than 5 %

3.7
compressive strength of masonry units (other than manufactured stone)
mean (not normalized) strength of a sample of masonry units tested in accordance with the test method prescribed in BS EN 772-1

3.8
column
isolated vertical loadbearing member the width of which is not more than four times its thickness

3.9
design load
characteristic load multiplied by a partial safety factor for loads

3.10
design strength
characteristic strength divided by a partial safety factor for material strength

3.11
effective height or length
height or length of a wall, pier or column assumed for calculating the slenderness ratio

3.12
effective thickness
thickness of a wall, pier or column assumed for calculating the slenderness ratio

3.13
key element
structural member capable of resisting a specified load

3.14
laterally loaded wall panels
walls subjected mainly to loads normal to the face of the wall

3.15
lateral support
<in relation to a wall or pier> support that will restrict movement in the direction of the thickness of the wall

3.16
lateral support
<in relation to a column> support that will restrict movement in the direction of its thickness or width

NOTE Lateral supports may be horizontal or vertical.

3.17
loadbearing wall
wall primarily designed to carry an imposed vertical load in addition to its own weight

3.18
masonry
assemblage of masonry units, either laid in-situ or constructed in prefabricated panels, in which the masonry units are bonded and solidly put together with mortar or grout

NOTE Masonry may be reinforced or unreinforced.

3.19
orthogonal ratio
ratio of the flexural strength of masonry when failure is parallel to the bed joints, to that when failure is perpendicular to the bed joints

3.20
pier
member which forms an integral part of a wall, in the form of a thickened section placed at intervals along the wall

3.21
slenderness ratio
ratio of the effective height or length to the effective thickness

3.22
masonry units
bricks or blocks of clay, calcium silicate, aggregate concrete, autoclaved aerated concrete, manufactured stone and natural stone

3.23
types of masonry mortar

3.23.1
designed masonry mortar
mortar, the composition and manufacturing method of which is chosen by the producer in order to achieve specified properties (performance concept)

3.23.2
prescribed masonry mortar
mortar made in pre-determined proportions, the properties of which are determined by the stated proportion of the constituents (recipe concept)

3.23.3
factory made masonry mortar
mortar batched and mixed in a factory

NOTE It may be "dry mortar" which is ready mixed only requiring the addition of water, or "wet mortar" which is supplied ready for use.

3.23.4
semi-finished masonry mortar
mortar defined in **3.23.5** or **3.23.6**

3.23.5
pre-batched masonry mortar
mortar, the constituents of which are wholly batched in a factory, supplied to the building site and mixed there according to the manufacturer's specification and conditions

3.23.6
premixed lime–sand masonry mortar
mortar, the constituents of which are wholly batched and mixed in a plant, and supplied to the building site where further constituents specified or provided by the factory (e.g. cement and water) are added

3.23.7
thin layer masonry mortar
designed mortar with a maximum aggregate size less than or equal to a prescribed figure

NOTE It is usually used in joints between 1 mm and 3 mm thick.

3.24
types of wall

3.24.1
single leaf wall
wall of bricks or blocks laid to overlap in one or more directions and set solidly in mortar

3.24.2
double-leaf (collar jointed) wall
two parallel single-leaf walls, with a space between not exceeding 25 mm, and so tied together as to result in common action under load

3.24.3
cavity wall
two parallel single-leaf walls, usually spaced at least 50 mm apart, and effectively tied together with wall ties, the space between being left as a continuous cavity or filled with a non-loadbearing material

3.24.4
grouted cavity wall
two parallel single-leaf walls, spaced at least 50 mm apart, effectively tied together with wall ties, and with the intervening cavity filled with fine aggregate concrete (grout), which may be reinforced, so as to result in common action under load

3.24.5
faced wall
wall in which the facing and backing are so bonded as to result in common action under load

3.24.6
veneered wall
wall having a facing which is attached to the backing, but not so bonded as to result in common action under load

3.25
wallette
small masonry panel constructed for test purposes

4 Symbols

For the purposes of this part of BS 5628, the following symbols apply:

A	horizontal cross-sectional area
$\underline{B}$	width of a bearing under a concentrated load
b	width of column
E_u	worst credible earth or water lateral load (see Clause **17**)
e_a	additional eccentricity due to deflection in walls
e_x	eccentricity at top of a wall
e_t	total design eccentricity in the mid-height region of a wall
e_m	the larger of e_x or e_t
F_k	characteristic load
F_m	average of the maximum loads carried by two test panels
F_t	tie force
f_k	characteristic compressive strength of masonry
f_{kx}	characteristic flexural strength (tension) of masonry
f_v	characteristic shear strength of masonry
f_{yt}	characteristic tensile strength of flat metal section (shear connector)
G_k	characteristic dead load
g_A	design vertical load per unit area
g_d	design vertical dead load per unit area
h	clear height of wall or column between lateral supports
h_a	clear height of wall between concrete surfaces or other construction capable of providing adequate resistance to rotation across the full thickness of a wall
h_L	clear height of wall to point of application of a lateral load
h_{ef}	effective height or length of wall or column
k	multiplication factor for lateral strength of axially loaded walls
K	stiffness coefficient
L	length
L_a	span in accidental calculation
N_s	number of storeys in a building
n	axial load per unit length of wall, available to resist an arch thrust
n_w	design vertical load per unit length of wall
p_{lim}	acceptance limit for compressive strength of units
p_o	specified compressive strength of units
p_u	mean compressive strength of units
Q_k	characteristic imposed load
q_{lat}	design lateral strength per unit area
r	width of flat metal section (shear connector)
t	overall thickness of a wall or column
t_{ef}	effective thickness of a wall or column
t_p	thickness of a pier
t_1	thickness of leaf 1 of a cavity wall
t_2	thickness of leaf 2 of a cavity wall
u	thickness of flat metal section (shear connector)

v_h	design shear stress
W_k	characteristic wind load
y_u	deflection of test wall in the mid-height region
Z	section modulus
α	bending moment coefficient for laterally loaded panels
β	capacity reduction factor for walls allowing for effects of slenderness and eccentricity
γ_f	partial safety factor for load
γ_m	partial safety factor for material
γ_{mv}	partial safety factor for material in shear
μ	orthogonal ratio

5 Alternative materials and methods of design and construction

Where materials and methods are used that are not referred to in this British Standard, their use is not discouraged, provided that the materials conform to the requirements of the appropriate British Standards or other documents, and that the methods of design and construction are such as to ensure a standard of strength and durability at least equal to that recommended in this British Standard.

Where materials or methods are used that are not referred to by this or any other British Standard, they should be proven by test, and the test assembly should be representative, as to materials, workmanship and details, of the design and construction for which approval is desired, and should be built under conditions truly representative of the conditions in the actual building construction. Details of the testing procedures to determine the characteristic compressive strength, the characteristic shear strength and characteristic flexural strength of masonry are set out in BS EN 1052-1, BS EN 1052-3 and BS EN 1052-2 respectively.

Section 2. Materials, components and workmanship

6 General

The materials, components and workmanship used in the construction of loadbearing walls should conform to the appropriate clause in BS 5628-3.

7 Masonry units

The following masonry units should conform to the appropriate clauses in the relevant British Standard:

clay masonry units	BS EN 771-1
calcium silicate masonry units	BS EN 771-2
aggregate concrete masonry units	BS EN 771-3
autoclaved aerated concrete masonry units	BS EN 771-4
manufactured stone masonry units	BS EN 771-5
natural stone masonry units	BS EN 771-6
bricks of special shapes and sizes	BS 4729

Materials that have been previously used should only be re-used in masonry construction after they have been thoroughly cleaned and conform to the code recommendations for similar new materials.

8 Laying of masonry units

8.1 General

Masonry units will normally be laid on their bed faces, but some concrete block walls will be built by laying blocks flat; tables for compressive strength are given for this circumstance. In other cases where units are used under load in another aspect, laid on stretcher or on end, the strength of the units should be determined for this aspect in accordance with BS 5628-2, Annex D or the test method prescribed in BS EN 772-1.

When bed joints are to be raked out for pointing, allowance should be made in design for the resulting loss of strength.

8.2 Bricks with frogs

Bricks should normally be laid on a full bed of mortar with the frog, or larger frog, uppermost, which should be filled with mortar as the work proceeds.

8.3 Hollow and cellular blocks

8.3.1 *Hollow clay blocks*

Hollow clay blocks conforming to BS EN 771-1 may have horizontal or vertical formed voids. The blocks should be laid on a full bed of mortar, except those with vertical perforations which are designed to be laid with shell bedding in accordance with the manufacturer's recommendations.

8.3.2 *Hollow and cellular concrete blocks*

Hollow and cellular concrete blocks conforming to BS EN 771-3 may be laid on a full bed of mortar. Occasionally such blocks are laid with shell bedding.

9 Rate of laying

The maximum height that should normally be built in a day is 1.5 m except when thin layer mortar is used.

10 Forming of chases and holes

Chasing of completed walls or the formation of holes should be carried out only when approved by the designer and then in accordance with BS 5628-3.

11 Damp proof courses

Damp proof courses should conform to the requirements of one of the British Standards, as appropriate, specified in BS 5628-3:2005, Clause **4.7**.

Designers should pay particular attention to the characteristics of the materials chosen for damp proof courses. Materials that squeeze out are undesirable in highly stressed walls, and the effect of sliding at the damp proof course should be considered especially in relation to lateral loading. In general, advice on the resistance to compression, tension, sliding and shear should be sought from the manufacturers of the damp proof course.

Methods of test for damp proof courses are prescribed in DD 86-1 and -2 and BS EN 1052-4.

12 Wall ties, tension straps, joist hangers and brackets

Wall ties, tension straps, joist hangers and brackets should conform to BS EN 845-1 and should be used in accordance with the recommendations made in BS 5628-3.

13 Cements

The following cements, or combination of cements, are suitable for use in mortars:

Cements:

Portland cement	BS EN 197-1 Notation CEM I
Limestone cement	BS EN 197-1 Notation CEM II/A-L and CEM II/A-LL
Sulfate-resisting Portland cement	BS 4027
Portland-slag cement	BS EN 197-1 Notation CEM or II/A-S or II/B-S
Portland-fly ash cement	BS EN 197-1 Notation CEM II/A-V or II/B-V
Masonry cement (inorganic filler, other than lime)	BS EN 413-1, Class MC
Masonry cement (lime)	BS EN 197-1 Notation CEM I (not less than 75 % by mass) and lime

Combinations of cements:

a) Combinations produced in the mortar mixer from Portland cement CEM I conforming to BS EN 197-1 and ground granulated blastfurnace slag conforming to BS 6699 where the proportions and properties conform to CEM II/A-S or CEM II/B-S of BS EN 197-1:2000, except Clause **9** of that standard.

b) Combinations produced in the mortar mixer from Portland cement CEM I conforming to BS EN 197-1 and limestone fines conforming to BS 7979 where the proportions and properties conform to CEM II/A-L or CEM II/A-LL of BS EN 197-1:2000, except Clause **9** of that standard.

c) Combinations produced in the mortar mixer from Portland cement CEM I conforming to BS EN 197-1 and pulverized fuel ash conforming to BS 3892-1, or to BS EN 450-1, where the proportions and properties conform to CEM II/A-V or CEM II/B-V of BS EN 197-1:2000, except Clause **9** of that standard.

The use of high alumina cement is not permitted.

14 Masonry mortars

14.1 General

The use of mortars should be in accordance with the recommendations given in BS 5628-3. For site made mortars, the mixing of the mortar should be in accordance with BS 5628-3. For factory made, semi-finished factory made and pre-batched masonry mortars, BS EN 998-2 applies.

Mortars should be designed or prescribed. For designed mortars, the compressive strength of the mortar provides the control of the hardened mortar quality. When samples are taken from a designed mortar in accordance with BS EN 1015-2, and tested in accordance with BS EN 1015-11, the compressive strength of the mortar should not be less than the declared compressive strength. Table 1 shows the relationship of compressive strength classes to strength.

14.2 Semi-finished factory made and pre-batched mortars

Semi-finished factory made and pre-batched mortars should conform to the requirements of BS EN 998-2. For designed mortars, the manufacturer should declare the strength that he is offering; for prescribed mortars, the provisions given in **14.1** apply.

Where pre-mixed lime-sand mortars are used, the specified addition of cement on site should be gauged.

14.3 Colouring agents

Pigments should conform to the requirements of BS EN 12878 and should not exceed 10 % by mass of the cement in the mortar. Care should be taken to ensure that the pigment is evenly distributed throughout the mortar and that the strength of the mortar remains adequate. Carbon black should be limited to 3 % by mass of the cement.

14.4 Plasticizers

Plasticizers should conform to the requirements of BS EN 934-3.

If plasticizers are used, it is important to ensure that the manufacturer's instructions about quantity and mixing time are carefully followed.

14.5 Frost inhibitors

The use of calcium chloride or frost inhibitors based on calcium chloride is not permitted in mortars.

Table 1 — Masonry mortars

	Mortar designation	Compressive strength class	Prescribed mortars (proportion of materials by volume) (see notes 1 and 2)				Compressive strength at 28 days
			cement[a]: lime: sand with or without air entrainment	cement[a]: sand with or without air entrainment	masonry cement[b]: sand	masonry cement[c]: sand	N/mm^2
↓ Increasing ability to accommodate movement, e.g. due to settlement, temperature and moisture changes	(i)	M12	1 : 0 to ¼ : 3	—	—	—	12
	(ii)	M6	1 : ½ : 4 to 4½	1 : 3 to 4	1 : 2½ to 3½	1 : 3	6
	(iii)	M4	1 : 1 : 5 to 6	1 : 5 to 6	1 : 4 to 5	1 : 3½ to 4	4
	(iv)	M2	1 : 2 : 8 to 9	1 : 7 to 8	1 : 5½ to 6½	1 : 4½	2

[a] Cement, or combination of cements, in accordance with Clause **13**, except masonry cements

[b] Masonry cement in accordance with Clause **13**, (inorganic filler other than lime)

[c] Masonry cement in accordance with Clause **13**, (lime)

NOTE 1 Proportioning by mass will give more accurate batching than proportioning by volume, provided that the bulk densities of the materials are checked on site.

NOTE 2 When the sand portion is given as, for example, 5 to 6, the lower figure should be used with sands containing a higher proportion of fines whilst the higher figure should be used with sands containing a lower proportion of fines.

Section 3. Design: objectives and general recommendations

15 Basis of design

The design of loadbearing masonry members should be undertaken primarily to ensure an adequate margin of safety against the ultimate limit state being reached. Generally, this is achieved by ensuring that the design strength of a member is greater than or equal to the design load. However, for some design situations, for example freestanding walls with no allowable flexural strength or axially loaded walls subject to lateral loads, the fundamental relationship which governs the design will be independent of the material strength and the partial safety factors associated with it.

The factor γ_m makes allowance for the variation in the quality of the materials and for the possible difference between the strength of masonry constructed under site conditions and that of specimens built in the laboratory for the purpose of establishing its physical properties.

The partial safety factors for loads (γ_f), used in this code for the ultimate limit state, are introduced to take account of:

a) possible unusual increases in load beyond those considered in deriving the characteristic load;

b) inaccurate assessment of effects of loading, and unforeseen stress redistribution within the structure;

c) the variations in dimensional accuracy achieved in construction.

The structural design should also ensure that there is an adequate margin of safety against a serviceability limit state being reached. In the case of the serviceability limit state of cracking due to axially applied service loads, an adequate margin may be assumed to exist when the design satisfies the ultimate limit state.

The risk of adverse effects on the structure, including non-loadbearing elements such as partitions, arising from expansion or contraction due to temperature and moisture changes, creep, settlement or deformation of flexural members, should be assessed. Where necessary, suitable details should be employed to maintain an adequate margin of safety against a limit state being reached.

16 Stability

16.1 General considerations

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

To ensure a robust and stable design it will be necessary to consider the layout of structure on plan, returns at the ends of walls, interaction between intersecting walls and the interaction between masonry walls and the other parts of the structure.

The design recommendations in section four assume that all the lateral forces acting on the whole structure are resisted by walls in planes parallel to these forces, or by suitable bracing. As well as the above general considerations, attention should be given to the following recommendations:

a) buildings should be designed to be capable of resisting a uniformly distributed horizontal load equal to 1.5 % of the total characteristic dead load (i.e. 0.015 G_k) above any level (see **18**b) and c));

b) connections of the type indicated in Annex C should be provided as appropriate at floors and roofs.

16.2 Earth retaining and foundation structures

The overall dimensions and stability of earth retaining and foundation structures (e.g. the area of pad footings, etc.) should be determined by appropriate geotechnical procedures, which are not considered in this British Standard. However, in order to establish section sizes and strengths that will give adequate safety and serviceability without undue calculation, it is appropriate in normal design situations to apply values of γ_f comparable to those applied to other forms of loading.

The partial safety factor for loads (γ_f) should be applied to all earth and water loads unless they derive directly from loads which have already been factored in alternative ways to those described in Clauses **17** and **18**, in which case the loads should be derived to achieve equilibrium with other design loads. When applying γ_f, no distinction is made between adverse and beneficial loads.

16.3 Accidental forces

In addition to designing the structure to support loads arising from normal use, there should be a reasonable probability that it will not collapse catastrophically under the effect of misuse or accident. No structure can be expected to be resistant to the excessive loads or forces that could arise due to an extreme cause, but it should not be damaged to an extent disproportionate to the original cause.

Furthermore, owing to the nature of a particular occupancy or use of a structure (e.g. flour mill, chemical plant, etc.), it may be necessary in the design concept or a design appraisal to consider the effect of a particular hazard and to ensure that, in the event of an accident, there is an acceptable probability of the structure remaining after the event, even if in a damaged condition.

Where there is the possibility of vehicles running into and damaging or removing vital loadbearing members of the structure in the ground floor, the provision of bollards, walls, retaining earth banks, etc. should be considered.

16.4 During construction

The designer should consider whether special precautions or temporary propping are necessary to ensure the overall stability of the structure or of individual walls during construction.

17 Loads

Ideally, the characteristic load on a structure should be determined statistically.

Since it is not yet possible to determine loads in this way, the following should be used as characteristic loads.

a) *Characteristic dead load*. The characteristic dead load G_k is the weight of the structure complete with finishes, fixtures and partitions and should be taken as equal to the dead load as defined in and calculated in accordance with BS 6399-1.

b) *Characteristic imposed load*. The characteristic imposed load Q_k should be taken as the imposed load as defined in and calculated in accordance with BS 6399-1 and BS 6399-3.

c) *Characteristic wind load*. The characteristic wind load W_k should be taken as the wind load calculated in accordance with BS 6399-2.

For the purposes of this standard, worst credible earth and water lateral loads, E_u, should be obtained in accordance with BS 8002 (see also Clause **18**).

18 Design loads: partial safety factor, γ_f

When using the design relationship for the ultimate limit state given in Sections 3 and 5, the design load should be taken as the sum of the products of the component characteristic loads multiplied by the appropriate partial safety factor, as shown below. Where alternative values are shown, that producing the more severe conditions should be selected.

a) Dead and imposed load

i) design dead load = 0.9 G_k or 1.4 G_k

ii) design imposed load = 1.6 Q_k

iii) design worst credible earth and water lateral load = 1.2 E_u

b) Dead and wind load

i) design dead load = 0.9 G_k or 1.4 G_k

ii) design wind load = 1.4 W_k

iii) design worst credible earth and water lateral load = 1.2 E_u

In the particular case of freestanding walls and laterally loaded wall panels, the removal of which would in no way affect the stability of the remaining structure, γ_f applied on the wind load may be taken as 1.2.

c) Dead, imposed and wind load

i) design dead load = 1.2 G_k

ii) design imposed load = 1.2 Q_k

iii) design wind load = 1.2 W_k

iv) design worst credible earth and water lateral load = 1.2 E_u

d) Accidental damage (see Clause **33**)

i) design dead load = 0.95 G_k or 1.05 G_k

ii) design imposed load = 0.35 Q_k except that, in the case of buildings used predominantly for storage, or where the imposed load is of a permanent nature, 1.05 Q_k should be used

iii) design wind load = 0.35 W_k

For all these cases:

G_k is the characteristic dead load;

Q_k is the characteristic imposed load;

W_k is the characteristic wind load;

E_u is the worst credible earth and lateral load (see Clause **17**)

and the numerical values are the appropriate γ_f factors.

Where other than worst credible earth and water lateral loads are used, such as nominal lateral loads determined in accordance with the *Civil Engineering Code of Practice No. 2* [3], the appropriate safety factor γ_f for the worst credible earth and water later load determination is 1.4.

In design, each of the load combinations a) to d) should be considered and that giving the most severe conditions should be adopted.

In certain circumstances, other values of γ_f may be appropriate, e.g. in farm buildings. Reference should be made to the relevant British Standards, e.g. BS 5502-22.

19 Characteristic compressive strength of masonry, f_k

19.1 Normal masonry

19.1.1 *General*

The characteristic compressive strength, f_k, of any masonry may be determined by tests on specimens, in accordance with BS EN 1052-1.

For normally bonded masonry (other than manufactured stone) defined in terms of the shape and compressive strength of the masonry units and the classification of the mortar (see Table 1), the values given in Table 2 may be taken to be the characteristic compressive strength, f_k, of walls constructed under laboratory conditions tested at an age of 28 days under axial compression in such a manner that the effects of slenderness may be neglected. Linear interpolation within the tables is permitted.

Table 2a) applies to masonry built with standard format bricks of clay or calcium silicate, conforming to the requirements of BS EN 771-1 or -2, and having no more than 25 % formed voids (perforations) or 20 % frogs.

Table 2b) applies to masonry built with autoclaved aerated concrete blocks having a ratio of height to least horizontal dimension of 0.6.

Table 2c) applies to masonry built with aggregate concrete blocks and having a ratio of height to least horizontal dimension of 0.6.

Table 2d) applies to masonry built with aggregate concrete blocks having not more than 25 % of formed voids and a ratio of height to least horizontal dimension of between 2.0 and 4.5.

Table 2e) applies to masonry built with autoclaved aerated concrete blocks having a ratio of height to least horizontal dimension of between 2.0 and 4.5.

Table 2f) applies to masonry built with aggregate concrete blocks having more than 25 % but less than 60 % of formed voids and a ratio of height to least horizontal dimension of between 2.0 and 4.5.

Table 2g) applies to masonry built with solid aggregate concrete blocks having a block height/wall thickness ratio of between 1.0 and 1.2 as a collar jointed wall.

Table 2h) applies to masonry built with solid aggregate concrete blocks laid flat having an as laid height/wall thickness ratio of between 0.4 and less than 0.6

The characteristic compressive strength of masonry bonded with thin layer mortar may be taken as the values given for mortar strength class M 12 (mortar designation (i)) in Table 2.

19.1.2 *Walls or columns of small plan area*

Where the horizontal cross-sectional area of a loaded wall or column is less than 0.2 m^2, the characteristic compressive strength should be multiplied by the factor:

$(0.70 + 1.5A)$

where A is the horizontal loaded cross-sectional area of the wall or column (in m^2).

19.1.3 *Narrow brick walls*

When brick walls are constructed so that the thickness of the wall or loaded inner leaf of a cavity wall is equal to the width of a standard format brick, the values of f_k obtained from Table 2a) may be multiplied by 1.15.

Table 2 — Characteristic compressive strength of masonry, f_k, in N/mm^2

a) — Constructed with standard format bricks of clay and calcium silicate having no more than 25% of formed voids, or 20% frogs

Mortar strength Class/Designation	Compressive strength of unit (N/mm²)[a]										
	5	10	15	20	30	40	50	75	100	125	150
M12 / (i)	2.5	4.0	5.3	6.4	8.3	10.0	11.6	15.2	18.3	21.2	23.9
M6 / (ii)	2.5	3.8	4.8	5.6	7.1	8.4	9.5	12.0	14.2	16.1	17.9
M4 / (iii)	2.5	3.4	4.3	5.0	6.3	7.4	8.4	10.5	12.3	14.0	15.4
M2 / (iv)	2.2	2.8	3.6	4.1	5.1	6.1	7.1	9.0	10.5	11.6	12.7

b) — Constructed with autoclaved aerated concrete blocks having a ratio of height to least horizontal dimension of 0.6

Mortar strength Class/Designation	Compressive strength of unit (N/mm²)[a]				
	2.9	3.6	5.2	7.3	10.4
M12 / (i)	1.4	1.7	2.5	3.4	4.4
M6 / (ii)	1.4	1.7	2.5	3.2	4.2
M4 / (iii)	1.4	1.7	2.5	3.2	4.1
M2 / (iv)	1.4	1.7	2.5	2.8	3.5

c) — Constructed with aggregate concrete blocks having a ratio of height to least horizontal dimension of 0.6

Mortar strength Class/Designation	Compressive strength of unit (N/mm²)[a]								
	2.9	3.6	5.2	7.3	10.4	17.5	22.5	30	40 or greater
M12 / (i)	1.4	1.7	2.5	3.4	4.4	6.3	7.5	9.5	11.2
M6 / (ii)	1.4	1.7	2.5	3.2	4.2	5.5	6.5	7.9	9.3
M4 / (iii)	1.4	1.7	2.5	3.2	4.1	5.1	6.0	7.2	8.2
M2 / (iv)	1.4	1.7	2.2	2.8	3.5	4.6	5.3	6.2	7.1

d) — Constructed with aggregate concrete blocks having not more than 25% of formed voids and a ratio of height to least horizontal dimension of between 2.0 and 4.5

Mortar strength Class/Designation	Compressive strength of unit (N/mm²)[a]								
	2.9	3.6	5.2	7.3	10.4	17.5	22.5	30	40 or greater
M12 / (i)	2.8	3.5	5.0	6.8	8.8	12.5	15.0	18.7	22.1
M6 / (ii)	2.8	3.5	5.0	6.4	8.4	11.1	13.0	15.9	18.7
M4 / (iii)	2.8	3.5	5.0	6.4	8.2	10.1	12.0	14.5	16.8
M2 / (iv)	2.8	3.5	4.4	5.6	7.0	9.1	10.5	12.5	14.5

e) — Constructed with autoclaved aerated concrete blocks having a ratio of height to least horizontal dimension of between 2.0 and 4.5

Mortar strength Class/Designation	Compressive strength of unit (N/mm²)[a]				
	2.9	3.6	5.2	7.3	10.4
M12 / (i)	2.8	3.5	5.0	6.8	8.8
M6 / (ii)	2.8	3.5	5.0	6.4	8.4
M4 / (iii)	2.8	3.5	5.0	6.4	8.2
M2 / (iv)	2.8	3.5	4.4	5.6	7.0

[a] Measured in normal direction of test for units.

Table 2 — Characteristic compressive strength of masonry, f_k, in N/mm^2 (*continued*)

f) — Constructed with aggregate concrete blocks having more than 25% but less than 60% of formed voids and a ratio of height to least horizontal dimension of between 2.0 and 4.5

Mortar strength Class/Designation	Compressive strength of unit (N/mm^2)[a]								
	2.9	**3.6**	**5.2**	**7.3**	**10.4**	**17.5**	**22.5**	**30**	**40 or greater**
M12 / (i)	2.8	3.5	5.0	6.6	8.1	11.2	13.1	16.0	19.4
M6 / (ii)	2.8	3.5	5.0	6.4	7.5	9.9	11.6	14.0	16.7
M4 / (iii)	2.8	3.5	5.0	6.1	7.1	9.0	10.2	12.0	14.0
M2 / (iv)	2.8	3.5	4.4	5.8	6.7	8.0	8.9	10.2	11.5

g) — Constructed with solid aggregate concrete blocks having a block height/wall thickness ratio of between 1.0 and 1.2 as a collar jointed wall

Mortar strength Class/Designation	Compressive strength of unit (N/mm^2)[a]								
	2.9	**3.6**	**5.2**	**7.3**	**10.4**	**17.5**	**22.5**	**30**	**40 or greater**
M12 / (i)	2.8	3.4	4.4	5.5	7.0	9.7	11.6	14.2	16.5
M6 / (ii)	2.5	2.8	3.5	4.5	5.7	7.9	9.4	11.6	13.5
M4 / (iii)	2.3	2.4	3.1	4.0	5.1	7.0	8.3	10.3	12.0
M2 / (iv)	2.2	2.3	2.5	3.2	4.1	5.7	6.8	8.2	9.5

h) — Constructed with solid aggregate concrete blocks laid flat having an as laid height/wall thickness ratio of between 0.4 and less than 0.6

Mortar strength Class/Designation	Compressive strength of unit (N/mm^2)[a]								
	2.9	**3.6**	**5.2**	**7.3**	**10.4**	**17.5**	**22.5**	**30**	**40 or greater**
M12 / (i)	2.8	3.5	4.5	5.6	7.2	9.7	11.3	13.5	15.4
M6 / (ii)	2.5	2.9	3.7	4.6	5.9	7.9	9.1	10.8	12.3
M4 / (iii)	2.3	2.5	3.2	4.1	5.2	7.0	8.1	9.6	11.1
M2 / (iv)	2.2	2.1	2.6	3.3	4.3	5.7	6.6	7.8	9.0

[a] Measured in normal direction of test for units.

19.1.4 ***Walls constructed with bricks having more than 25% and less than 55% of formed voids***

The characteristic compressive strength of masonry of walls constructed with bricks of clay or calcium silicate that have more than 25% and less than 55% of formed voids (perforations), may be taken from the values given in Table 2a) multiplied by 0.8, provided that the compressive strength of the units does not exceed 55 N/mm^2.

19.1.5 ***Walls constructed in modular bricks***

For 90 mm wide × 90 mm high modular bricks conforming to the requirements of BS EN 771-1, -2 and -3, the values of f_k given in Table 2a) and Table 2c) as relevant may be multiplied by the following factors:

a) masonry thickness equal to width of brick: 1.25;

b) other thickness: 1.10.

19.1.6 ***Walls constructed of wide bricks***

When walls are constructed with bricks having a ratio of height to least horizontal dimension of less than 0.6, the value of f_k should be obtained from tests carried out in accordance with BS EN 1052-1.

19.1.7 ***Hollow concrete block walls***

When walls are built of blocks having more than 25% and less than 60% of formed voids and a ratio of height to least horizontal dimension of between 0.6 and 2.0, the value of f_k should be obtained by interpolation between the values given in Table 2c) and those in Table 2f).

19.1.8 ***Concrete block walls having not more than 25% formed voids***

When walls are built of concrete blocks having not more than 25% of formed voids and having a ratio of height to least horizontal dimension of between 0.6 and 2.0, the value of f_k should be obtained by interpolation between the values given in Table 2c) and those in Table 2e).

19.1.9 ***Walls of hollow concrete blocks filled with in-situ concrete***

When walls are built with hollow concrete blocks and the vertical cavities are completely filled with in-situ concrete, the characteristic compressive strength of the masonry should be obtained as if the blocks had not more than 25% formed voids (see **19.1.8**), provided that:

a) the compressive strength of the blocks is assessed on their net area;

b) the 28-day cube strength of the concrete infilling is not less than the compressive strength of the blocks derived from a) above.

19.1.10 ***Natural stone masonry***

The characteristic compressive strength of stone masonry with bed joints 10 mm thick or less, and in mortar designation (iii) or stronger, may be taken as 0.35 times the mean compressive strength of the natural stone masonry units or representative cubes when prepared in accordance with BS EN 771-6 and tested in accordance with BS EN 772-1. Where the strength is derived from cubes, consideration should be given to the direction of any bedding planes, see BS EN 771-6.

19.1.11 ***Random rubble masonry***

The characteristic strength of random rubble masonry may be taken as 75% of the corresponding strength of natural stone masonry built with similar materials. In the case of rubble masonry built with lime mortar, the characteristic strength may be taken as one-half of that for masonry in mortar strength class M2 (designation (iv)).

19.2 Masonry units laid other than on the normal bed face

Where masonry units, excluding aggregate concrete blocks, are laid other than on the normal bed face, the compressive strength of the unit as determined for that direction (see Clause **8**) should be adopted when using Table 2. When aggregate concrete blocks are laid on their stretcher face, that is laid flat, Table 2h) should be used in conjunction with the declared compressive strength of the unit when tested in the normal direction.

19.3 Perforated bricks and hollow blocks

The compressive strength of bricks and blocks having formed voids in them (perforated bricks and hollow blocks) is determined by dividing the ultimate load by the gross plan area of the unit, as if it were solid. This compressive strength should therefore be used in obtaining the value of f_k from Table 2a) to Table 2f). Hollow concrete blocks are sometimes laid with mortar on the two outer strips of block (shell bedding), in which case the value of f_k should be obtained as usual from interpolation between Table 2c) and Table 2f), but the design strength of the wall should be reduced by the ratio of the bedded area to the net area of block.

20 Characteristic flexural strength of masonry, f_{kx}

20.1 General

The characteristic flexural strength, f_{kx}, should be used only in the design of masonry in bending. In general, no direct tension should be allowed in masonry. However, at the designer's discretion, direct tension up to half the values given in Table 3 for the plane of failure parallel to the bed joints may be allowed when suction forces arising from wind loads on roof structures are transmitted to masonry walls, or when the probable effects of misuse or accidental damage (see Section 5) are being considered. In no circumstances may the combined flexural and direct tensile stresses exceed the values given in Table 3.

Flexural tension should be relied on at a damp proof course only if the damp proof course consists of a material that has been proved by tests (see DD 86-1) to permit the joint to transmit tension, or if it is constructed with bricks conforming to the requirements of BS EN 771-1 for damp proof course bricks.

20.2 Flexural strength

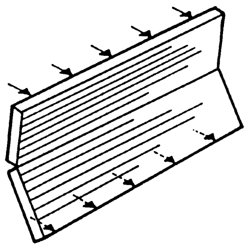
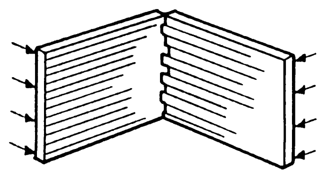
The characteristic flexural strength values given in Table 3 may be used for the categories of brick, block and mortar shown or, alternatively, tests may be carried out in accordance with BS EN 1052-2.

Linear interpolation between entries in Table 3 is permitted for:

a) concrete block walls of thickness between 100 mm and 250 mm;

b) concrete blocks of compressive strength between 2.9 N/mm^2 and 7.3 N/mm^2 in a wall of given thickness.

Table 3 — Characteristic flexural strength of masonry, f_{kx}, N/mm^2

	Plane of failure parallel to bed joints 			**Plane of failure perpendicular to bed joints** 		
Mortar strength class/designation	M12 (i)	M6 and M4 (ii) and (iii)	M2 (iv)	M12 (i)	M6 and M4 (ii) and (iii)	M2 (iv)
Clay bricks having up to 40% of formed voids and 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	0.35	1.5	1.1	1.0
over 12% (see Note 1)	0.4	0.3	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 (solids or hollow) of compressive strength (in N/mm^2): (see Note 2)						
2.9 3.6 7.3 — used in walls of thickness up to 100 mm (see Note 3)	0.25		0.2	0.40 0.45 0.60		0.4 0.4 0.5
2.9 3.6 7.3 — used in walls of thickness 250 mm or greater (see Note 3)	0.15		0.1	0.25 0.25 0.35		0.2 0.2 0.3
10.4 17.5 and over — used in walls of any thickness (see Note 3)	0.25		0.2	0.75 0.90 (see Note 4)		0.60 0.7 (see Note 4)

NOTE 1 Tests to determine the water absorption of clay bricks should be performed in accordance with BS EN 772-7.

NOTE 2 Tests to determine the compressive strength of concrete blocks should be performed in accordance with BS EN 772-1.

NOTE 3 The thickness should be taken to be the thickness of the wall, for a single-leaf wall, or the thickness of the leaf, for a cavity wall.

NOTE 4 When used with flexural strength in the parallel direction, assume the orthogonal ratio $\mu = 0.3$.

NOTE 5 The characteristic flexural strength of masonry bonded with thin layer mortar may be taken as the values given for mortar strength class M12 (mortar designation (i)).

NOTE 6 See also Clause **19.2**.

21 Characteristic shear strength of masonry, f_v

21.1 General

21.1.1 *Horizontal direction*

The characteristic shear strength of masonry in the horizontal direction of the horizontal plane (see Figure 1) is given by:

$$f_v = f_{vko} + 0.6g_A$$

where

f_{vko} is the characteristic initial shear strength in N/mm²; and,

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 specified in Clause **18**.

f_v should be taken as not greater than 1.75 N/mm² for masonry built in thin layer mortar and mortar strength classes M12 and M6 / designations (i) and (ii) or 1.4 N/mm² for masonry built in mortar strength classes M4 and M2 / designations (iii) and (iv).

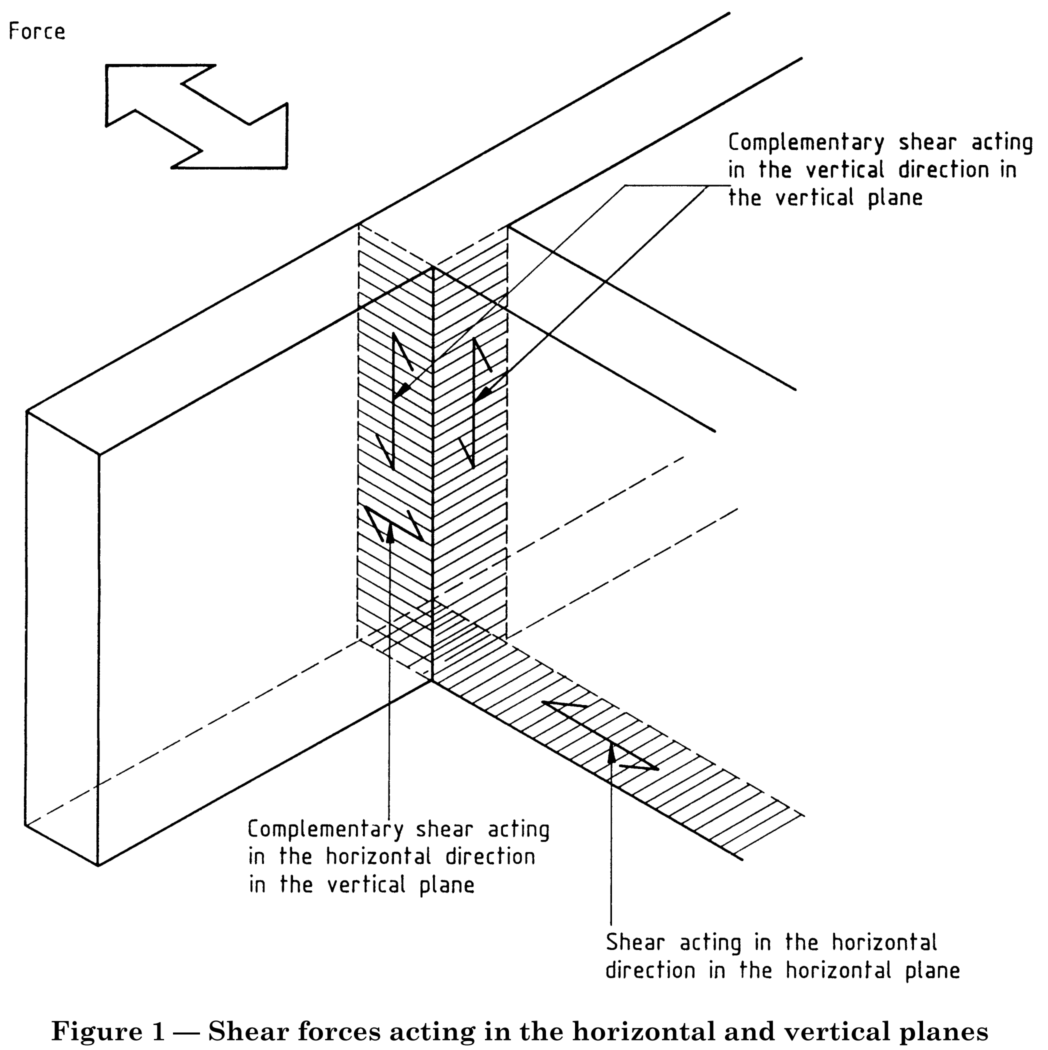


Figure 1 — Shear forces acting in the horizontal and vertical planes

21.1.2 ***Characteristic initial shear strength of masonry, f_{vko}***

The characteristic initial shear strength of masonry, f_{vko}, may be:

a) determined by tests in accordance with BS EN 1052-3;

b) taken as 0.35 N/mm^2 with clay/cal silicate units having less than 40% formed voids and concrete units having less than 50% formed voids for masonry built in thin layer mortar and mortar strength classes M12 and M6 / designations (i) and (ii); or

c) taken as 0.15 N/mm^2 for masonry built in mortar strength classes M4 and M2 / designations (iii) and (iv).

21.2 Vertical direction

The characteristic shear strength f_v of bonded masonry in the vertical direction of the vertical plane (see Figure 1) may be taken as:

a) for brick:

1) 0.7 N/mm^2 (for mortar strength classes M12 and M6 / designations (i) and (ii));

2) 0.5 N/mm^2 (for mortar strength classes M4 and M2 / designations (iii) and (iv));

b) for dense aggregate solid concrete block with a minimum strength of 7 N/mm^2:

0.35 N/mm^2 (for mortar strength classes M12, M6 and M4 / designations (i), (ii) and (iii)).

22 Coefficient of friction

The coefficient of friction between clean concrete and masonry faces may be taken as 0.6.

23 Partial safety factors for material strength, γ_m

23.1 General

The value of γ_m to be used in the application of the design procedures given in sections four and five should be related to the quality control exercised.

23.2 Quality control

The value of γ_m adopted should be commensurate with the degree of control exercised both during the manufacture of the masonry units and in the site supervision and also with the quality of the mortar used during construction. Two levels of control are recognized in each case, as detailed in **23.2.1** and **23.2.2**.

23.2.1 ***Manufacturing control***

The manufacturer of masonry units will declare whether the units are Category I or Category II according to BS EN 771-1 to -6.

23.2.2 ***Construction control***

23.2.2.1 *Normal category*

Normal category should be assumed whenever the work is carried out in accordance with the recommendations for workmanship made in BS 5628-3 or BS 8000-3, including appropriate supervision and inspection.

23.2.2.2 *Special category*

Special category of construction control may be assumed where the requirements of the 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 4;

b) the mortar conforms to BS EN 998-2 if it is factory made mortar, or,

if it is site mixed mortar, preliminary compression strength tests performed on the mortar to be used, in accordance with **A.1**, indicate conformity to the strength requirements in Table 1, and regular testing of the mortar used on site, in accordance with **A.1**, should show that the strength requirements of Table 1 are being maintained.

23.3 Values of γ_m for normal and accidental loads

The value of γ_m to be used in Clause **28** and Clause **30** should be obtained from Table 4. Table 4 also gives the values of γ_m to be used, except where otherwise stated, in respect of flexural stresses discussed in Clause **32**. When considering the probable effects of misuse or accident (see Clause **33**), the values given in Table 4 may be halved, except where otherwise required in **33.2**.

Table 4 — Partial safety factors for material strength

	Category of masonry units	Category of construction control	
		Special	Normal
Compression, γ_m	Category I	2.5	3.1
	Category II	2.8	3.5
Flexure, γ_m	Category I and II	2.5	3.0

Where tests in accordance with BS EN 1052-1, -2 and -3 have been performed, the γ_m factors to be applied to the characteristic strengths so determined may be taken as 0.9 times the values given in Table 4, provided the masonry conforms in all respects to the recommendations made in this British Standard.

Where the specimen tested incorporates constructional details or units not covered by this code, a similar procedure may be applied but the characteristic strength so obtained should be used only to design walls that incorporate the same constructional details or units.

23.4 Values of γ_{mv} for shear loads

The partial safety factor for masonry strength in shear, γ_{mv}, should be taken as 2.5 when mortar not weaker than strength class M2 (designation (iv)) is used. When considering the probable effects of misuse or accident (Section 5) the value of γ_{mv} may be reduced to 1.25.

23.5 Values of γ_{mv} for use with ties

The partial safety factor to be applied to the strength of wall ties (see **33.3**) conforming to BS EN 845-1 should be 3.0. When considering the probable effects of misuse or accidental damage, this value may be halved.

Section 4. Design: detailed considerations

24 Consideration of slenderness of walls and columns

24.1 Slenderness ratio

The slenderness ratio should not exceed 27, except in the case of walls less than 90 mm thick, in buildings of more than two storeys, where it should not exceed 20.

24.2 Lateral support

24.2.1 ***General***

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.

24.2.2 ***Horizontal or vertical lateral supports***

Horizontal or vertical lateral supports should be capable of transmitting to the elements of construction that provide lateral stability to the structure as a whole, the sum of the following design lateral forces:

a) the simple static reactions to the total applied design horizontal forces at the line of 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; the elements of construction that provide lateral stability to the structure as a whole need not be designed to support this force.

However, the designer should satisfy himself that loads applied to lateral supports will be transmitted to the elements of construction providing stability, e.g. by the floors or roofs acting as horizontal girders.

For wall ties, tension straps, joist hangers and brackets conforming to BS EN 845-1, the design resistance should be taken as the declared value of the relevant performance characteristic valid for the mortar and masonry units, divided by the value of γ_m for use with ties (see **23.5**).

For wall ties, the relevant performance characteristic may be compressive load capacity, tensile load capacity, shear load capacity in the vertical and horizontal direction. For tension straps the relevant performance characteristic is tensile load capacity and for joist hangers and brackets it is vertical load capacity. In each case the declared value should be one that is valid for the strength and type of mortar and the strength of the masonry units to be used.

24.2.3 ***Horizontal lateral supports***

24.2.3.1 Simple resistance to lateral movement may be assumed in the case of houses of not more than three storeys where the connections between structural elements conform to BS 8103-1. In all other cases, including buildings of more than three storeys, a connection capable of providing simple resistance to lateral movement may be assumed where connections are of the form illustrated in Annex C.

24.2.3.2 Enhanced resistance to lateral movement may be assumed where:

a) floors or roofs of any form of construction span on to the wall or column from both sides at the same level;

b) 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 less than 90 mm;

c) in the case of houses of not more than three storeys, a timber floor spans on to a wall from one side and has a bearing of not less than 90 mm.

It is preferable that columns be provided with lateral support in both horizontal directions.

24.2.4 ***Vertical lateral supports***

24.2.4.1 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 ten times the thickness of the supported wall or loadbearing leaf and is connected to it by metal anchors calculated in accordance with **24.2.2** and evenly distributed throughout the height at not more than 300 mm centres.

24.2.4.2 Enhanced resistance to lateral movement may be assumed where an intersecting or return wall as described in **24.2.4.1** is properly bonded to the supported wall or loadbearing leaf of a cavity wall.

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

24.3 Effective height or length

24.3.1 ***General***

The effective height or length of a loadbearing wall or column should be assessed taking account of the relative stiffness of the elements of structure connected to the wall or column and the efficiency of the connections. In the absence of detailed calculations, the designer may take the effective height or length from **24.3.2** or **24.3.3**.

24.3.2 ***Effective height***

24.3.2.1 *Walls*

The effective height of a wall may be taken as:

a) 0.75 times the clear distance between lateral supports which provide enhanced resistance to lateral movement; or

b) he clear distance between lateral supports which provide simple resistance to lateral movement.

24.3.2.2 *Columns*

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.

24.3.2.3 *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.

a) 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.

b) 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.

24.3.2.4 *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.

NOTE 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.

24.3.3 ***Effective length***

The effective length of a wall may be taken as:

a) 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;

b) 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.

24.4 Effective thickness

24.4.1 *General*

The effective thickness of a wall, column or pier is given in **24.4.2** and **24.4.3** and is illustrated in Figure 2.

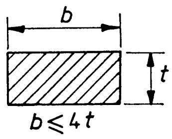
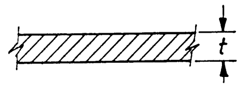
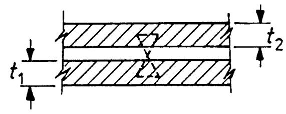
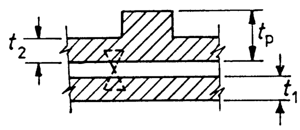
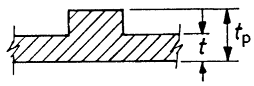
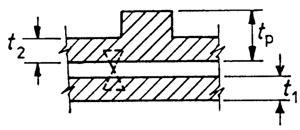
Type of wall:	**Column**	**Single-leaf wall**	**Cavity wall**
Plan shapes:			
Effective thickness:	t or b, depending on direction of bending	t	the greatest of: (a) 2/3 ($t_1 + t_2$); or (b) t_1; or (c) t_2
	Walls stiffened by piers		
Type of wall:	**Grouted cavity wall**	**Single-leaf**	**Cavity**
Plan shapes:			
Effective thickness:	$t_1 + t_2 + tc \leqslant 100$	$t \times K$	the greatest of: (a) 2/3 $t_1 + Kt_2$); or (b) t_1; or (c) Kt_2
		Where K is the stiffness coeffecient from Table 5	

Figure 2 — Effective thickness of columns and walls

24.4.2 *Walls and columns not stiffened by piers or intersecting walls*

For single leaf walls and columns the effective thickness is the actual thickness.

For cavity walls and columns the effective thickness should be taken as two-thirds the sum of the actual thicknesses of the two leaves or the actual thickness of the thicker leaf, whichever is the greater.

24.4.3 *Walls stiffened by piers or intersecting walls*

Where a wall, which may be one leaf of a cavity wall, is stiffened by piers, the effective thickness, t_{ef}, of the wall, or leaf of a cavity wall, is:

$$t_{ef} = t \times K$$

where

t is the actual thickness of the wall or leaf;

K is the appropriate stiffness coefficient taken from Table 5.

For a wall stiffened by intersecting walls, the appropriate stiffness coefficient may be determined from Table 5 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.

Table 5 — Stiffness coefficient for walls stiffened by piers

Ratio of pier spacing (centre to centre) to pier width	Ratio t_p/t 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 5 is permissible, but not extrapolation outside the limits given.			

25 Special types of wall

25.1 General

With all types of loadbearing wall which combine different materials, the possibility of overloading due to differential movement of the materials should be taken into account.

Where parts of a wall are not loadbearing, care may need to be taken to prevent the transfer of load to those parts due to movement of the loadbearing elements.

25.2 Cavity walls

25.2.1 *General*

Where 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 (see **19.1.2** and **24.2**).

25.2.2 *Minimum thickness of leaves*

Each leaf of a cavity wall should be not less than 75 mm thick.

25.2.3 *Width of cavity*

The width of the cavity may vary between 50 mm and 300 mm but should not be greater than 75 mm where either of the leaves is less than 90 mm in thickness. In special circumstances and with appropriate supervision, the width of the cavity may be reduced below 50 mm (but see also BS 5628-3).

25.2.4 *Selection and strength of wall ties*

The leaves of a cavity wall should be tied together by wall tie of a type that is suitable for the type of construction. The wall ties should be either embedded in the horizontal mortar joints at the time the units are laid or fixed in accordance with the manufacturer's instructions.

The design compressive and tensile resistance of the wall ties relevant to the nominal cavity width and design embedment length or fixings should exceed the design loads to which they will be subjected in respect of the nominal cavity width. Table 6 gives guidance on the selection of ties for normal application.

NOTE The wall tie types 1, 2, 3 and 4 and guidance on their use are given in Annex C.3.

25.2.5 *Density and positioning of wall ties*

The density (number of wall ties per square metre) should be not less than 2.5 ties/m^2 for walls in which both leaves are 90 mm or thicker and not less than 4.9 ties/m^2 for walls in which either leaf is less than 90 mm thick. Wall ties should be evenly distributed over the wall area, except around openings, and should preferably be staggered. At the vertical edges of openings and at vertical unreturned or unbonded edges (for example at movement joints and up the sloping verge of gable walls), additional wall ties should be used at a rate of one tie per 300 mm height or equivalent, placed not more than 225 mm from the edge.

25.2.6 *Embedment of wall ties*

The minimum embedment of a wall tie in the mortar joint should not be less than 50 mm in each leaf.

The length of the wall ties should be sufficient to give the minimum embedment having regard to normal site tolerances for cavity width and centring of the tie. Suitable minimum lengths are given in Table 6.

Table 6 — Selection of wall ties: types, categories and lengths

Least leaf thickness (one or both) mm	Nominal cavity width mm	Tie length mm	Wall tie user categories[c]
75	75 or less	200[a]	Types 1, 2, 3 or 4, selected on the basis of the design loading and design cavity width
90	76 to 100	225[a]	
90	101 to 125	250[a]	
90	126 to 150	275[a]	
90	151 to 175	300[a]	
90	176 to 300	[b]	

[a] This column gives the tie lengths, in 25 mm increments, that best meet the performance requirement that the embedment depth will not be less than 50 mm in both leaves, after taking into account all building and material tolerances, but also that the ties should not protrude from the face. Where the embedment depth needs to exceed 50 mm in both leaves (see **25.2.6**) the tie length may need to be increased accordingly.

[b] For cavities wider than 175 mm, and for a 50 mm embedment depth, calculate the length as the nominal cavity width plus 125 mm and select the nearest stock length. For an embedment depth exceeding 50 mm (see **25.2.6**) increase the calculated length accordingly.

[c] The strength and stiffness of masonry/masonry ties in accordance with Annex C ranges from Type 1, usually the stiffest, to Type 4, usually the least stiff.

25.3 External cavity walls

25.3.1 *General*

In order to avoid detrimental effects due to differential vertical movement between the inner and outer leaves, the designer may either:

a) limit the uninterrupted height of the outer leaf of external cavity walls; or

b) calculate the likely differential vertical movement of the outer masonry leaf with respect to the inner leaf and accommodate this movement by using suitable details.

25.3.2 *Limitation on uninterrupted height*

When the method of limiting the uninterrupted height is adopted, the outer leaf should be supported at intervals of not more than every third storey or every 9 m, whichever is less. However, for buildings not exceeding four storeys or 12 m in height, whichever is less, the outer leaf may be uninterrupted for its full height.

25.3.3 *Accommodation of differential vertical movement*

When the method of accommodating the differential vertical movement is adopted, the calculation should take into account all the factors such as elastic movement, moisture movement, thermal movement, creep, etc. The calculated differential movement should not exceed 30 mm. The details used should involve separate lintels for outer and inner leaves, the use of movement-tolerant wall ties, the fixing of windows to outer leaves, the provision of soft joints under sills and the use of suitable wall head details.

25.3.4 *Accommodation of horizontal movements*

Whichever of the above alternatives is adopted for differential vertical movement, consideration should also be given to the provision of vertical joints to accommodate horizontal movements (see BS 5628-3).

25.4 External walls of framed structures

Where a masonry wall is external to a structural frame the recommendations for avoidance of detrimental effects should be as given in **25.3.3**. When the wall is of bonded masonry construction, or is collar jointed, consideration should be given to the compatibility of the movement characteristics of the masonry units (see also **25.5**).

25.5 Faced walls

Where a wall is constructed of two different types of masonry unit, it should be designed in the same manner as a wall having the same total thickness and constructed entirely in the weaker unit, or as a veneered wall. The different types of unit should be compatible.

25.6 Veneered walls

While the dead weight of the veneer has to be included in the load calculated, any structural effect of the veneer should be neglected.

25.7 Double-leaf (collar jointed) walls

Where a wall is constructed of two separate leaves with a vertical collar joint not exceeding 25 mm wide between them it may be designed as either:

a) a cavity wall; or,

b) a single leaf wall, provided that the following conditions are satisfied.

1) Each leaf is at least 90 mm thick.

2) For concrete blockwork, the characteristic compressive strength is obtained from Clause **19**.

3) If the two leaves of the wall are constructed of different materials, it is designed on the assumption that the wall is constructed entirely in the weaker unit. The possibility of differential movement should be taken into account.

4) The load is applied to the two leaves and the eccentricity does not exceed $0.2t$ (except in the case of laterally loaded panels) where t is the overall thickness of the wall.

5) Shear ties conforming to BS EN 845-1 with a declared shear load capacity, relevant for the type of mortar and masonry units, of not less than 1.5 kN, or prefabricated bed joint reinforcement of equivalent performance, are provided at centres not exceeding 450 mm vertically and 900 mm horizontally.

6) The minimum embedment of the ties into each leaf is as recommended in **25.2.6**.

25.8 Grouted cavity walls

Where a wall is constructed of two separate leaves separated by a space of between 50 mm and 100 mm filled with concrete of 28-day strength not less than that of the mortar, and otherwise conforms to the requirements detailed in **25.7**b) 1), 3), 4), 5) and 6), it may be designed as a single leaf wall. The effective thickness should be taken as equal to the actual overall thickness.

26 Eccentricity in the plane of the wall

The eccentricity in the plane of a single wall can be calculated from statics alone. Where a horizontal force is resisted by several walls it may be distributed between the walls in proportion to their flexural stiffness about an axis perpendicular to the direction of the force. The forces in the walls may be determined by an appropriate elastic analysis. Connections for transmitting the horizontal force to the walls should be properly designed.

27 Eccentricity at right angles to the wall

Preferably, eccentricity of loading on walls and columns should be calculated but, at the discretion of the designer, 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 total vertical load on a wall is axial immediately above a lateral support.

28 Walls and columns subjected to vertical loading

28.1 Loads eccentric in the plane of the wall

Where the vertical resultant of all the loads acts eccentrically in the plane of the wall, the intensity of loading at any position should be assessed on the basis of the load distribution shown in Figure 3 and the wall strength should be calculated in accordance with **28.2**.

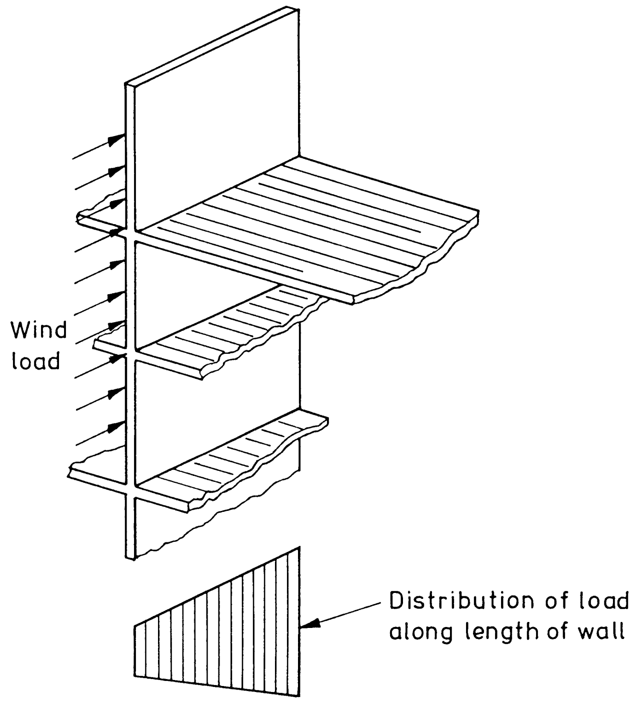


Figure 3 — Load distribution from loading eccentric to plane of wall

28.2 Design strength of masonry

28.2.1 *General*

The design strength of masonry is the characteristic strength multiplied by a capacity reduction factor, β, and divided by the appropriate partial safety factor for material.

28.2.2 *Design vertical load resistance of walls*

The design vertical load resistance of a wall per unit length is given by:

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

where

β	is a capacity reduction factor allowing for the effects of slenderness and eccentricity, and is obtained from Table 7;
f_k	is the characteristic strength of the masonry obtained from Clause **19**;
γ_m	is the appropriate partial safety factor for the material obtained from Table 4;
t	is the thickness of the wall.

Table 7 — Capacity reduction factor, β

Slenderness ratio h_{ef}/t_{ef}	Eccentrcity at top of wall, e_x			
	Up to 0.5t (see Note 1)	0.1t	0.2t	0.3t
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		

NOTE 1 It is not necessary to consider the effects of eccentricities up to and including 0.05t.

NOTE 2 Linear interpolation between eccentricities and slenderness ratios is permitted.

NOTE 3 The derivation of β is given in Annex B.

28.2.3 ***Design vertical load resistance of columns***

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

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

where

b	is the width of the column;
t	is the thickness of the column;

all other symbols are as given in **28.2.2**.

The value of β should be chosen as follows:

a) when the eccentricities about the major and minor axes at the top of the column are less than $0.05b$ and $0.05t$ respectively, from the second column of Table 7, basing the slenderness ratio on the value of t_{ef} appropriate to the minor axis;

b) when the eccentricities about the major and minor axes are less than $0.05b$ but greater than $0.05t$ respectively, from Table 7, using the values of eccentricity and slenderness ratio appropriate to the minor axis;

c) when the eccentricities about the major and minor axes are greater than $0.05b$ but less than $0.05t$ respectively, from Table 7, using the value of eccentricity appropriate to the major axis and the value of slenderness ratio appropriate to the minor axis or from Annex B deriving additional eccentricities about both axes;

d) when the eccentricities about the major and minor axes are greater than $0.05b$ and $0.05t$ respectively, from Annex B, deriving additional eccentricities about both axes.

28.2.4 ***Design vertical load resistance of cavity walls and columns***

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 in accordance with **28.2.2** or **28.2.3** as appropriate, in which case t_{ef} in Table 7 is the effective thickness of the cavity wall or column.

29 Walls subjected to shear forces

Where walls resist, in shear, horizontal forces acting in their plane, provision against the ultimate limit state in shear being reached may be assumed if the following relationship is satisfied:

$$v_h \leq \frac{f_v}{\gamma_{mv}}$$

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;
γ_{mv}	is the partial safety factor for material strength in shear (**23.4**);
f_v	is the characteristic shear strength of the masonry (Clause **21**).

30 Concentrated loads: stresses under and close to a bearing

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 either that the element applying the load is sensibly rigid, or that 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 Figure 4c), and dispersed in two planes within a zone contained by lines extending downwards at 45° from the edges of the loaded area.

The effect of the local load combined with stresses due to other loads (see Figure 5a) should be checked:

a) at the bearing, assuming a local design bearing strength of $1.25f_k/\gamma_m$ in the case of bearing type 1 (Figure 4a) or $1.5f_k/\gamma_m$ in the case of bearing type 2 (Figure 4b);

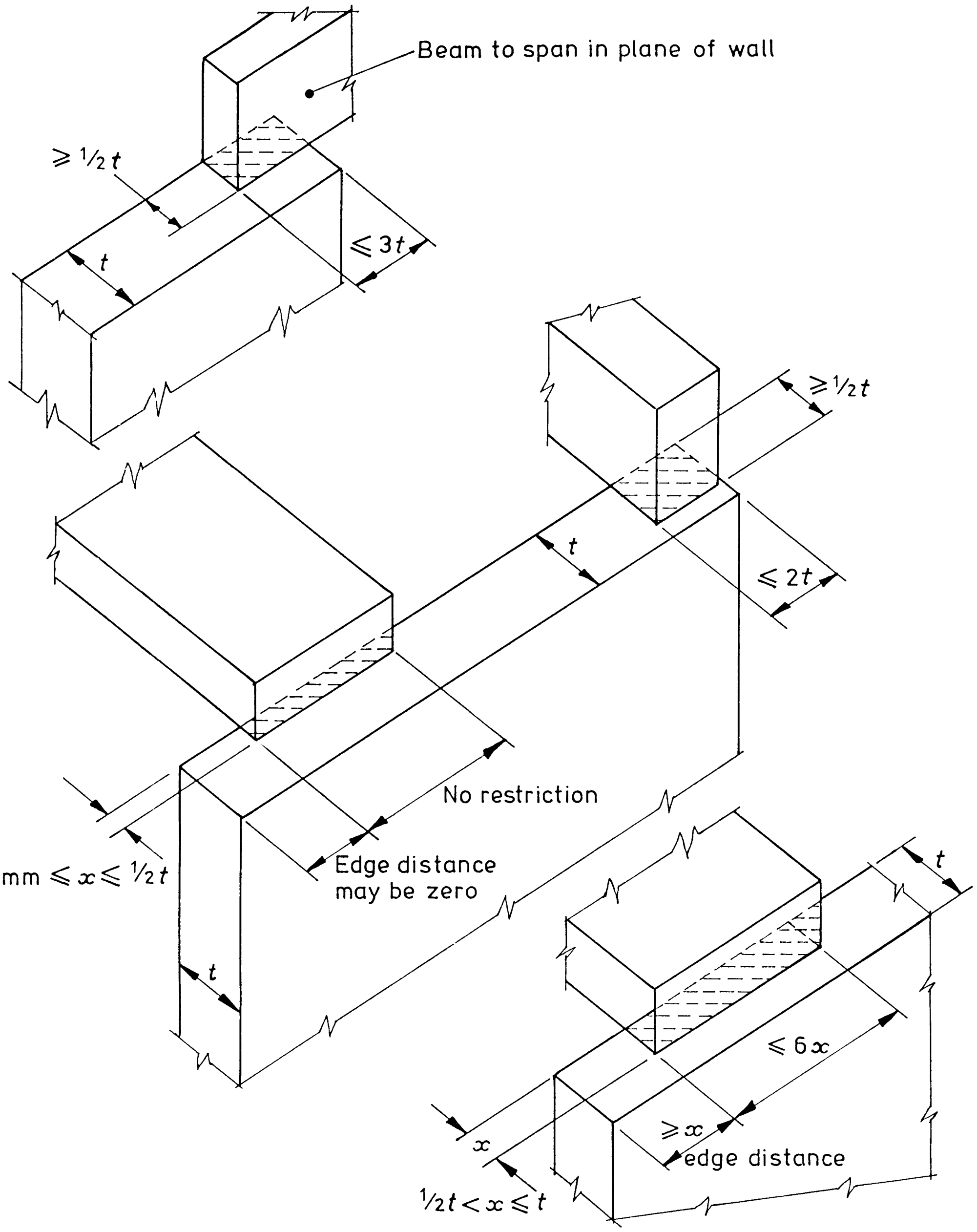
b) at a distance of 0.4 h below the bearing where the design strength should be calculated in accordance with Clause **28**.

where

f_k	is the characteristic strength of the masonry;
γ_m	is the appropriate partial safety factor for the material (see Table 4);
h	is the clear height of the wall;
β	is the capacity reduction factor from Table 7.

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 its plane (see Figure 4c), the maximum stress at the bearing combined with stresses due to other loads should not exceed $2.0f_k/\gamma_m$.

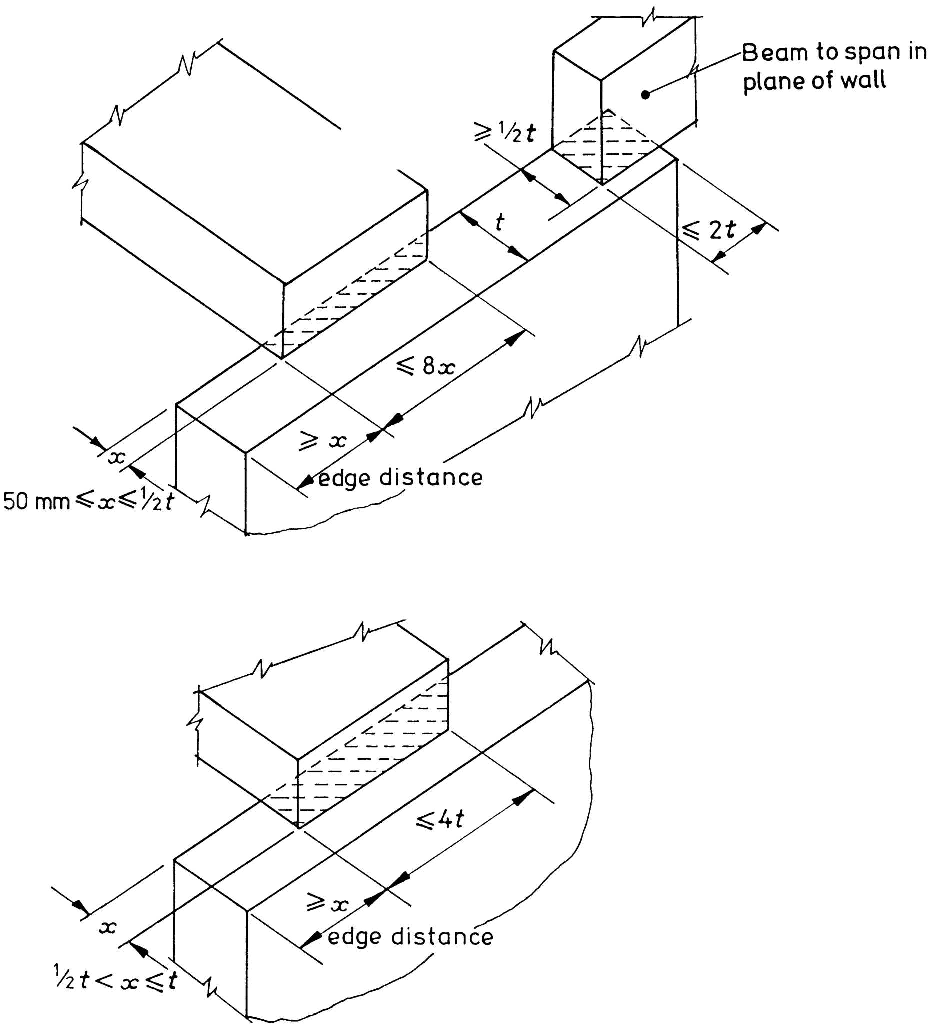
In this case, when checking the stress at a distance of $0.4h$ below the bearing, the design strength should be calculated in accordance with Clause **28** (see Figure 5b).



Local design strength $\frac{1.25 f_k}{\gamma_m}$

(a) Bearing type 1

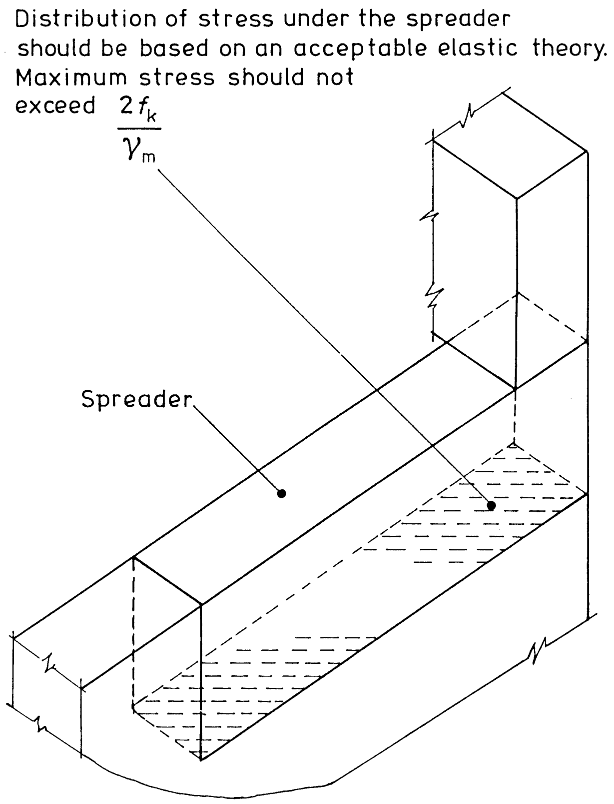
Figure 4 — Concentrated loads: types of bearing



Local design strength $\frac{1.5 f_k}{\gamma_m}$

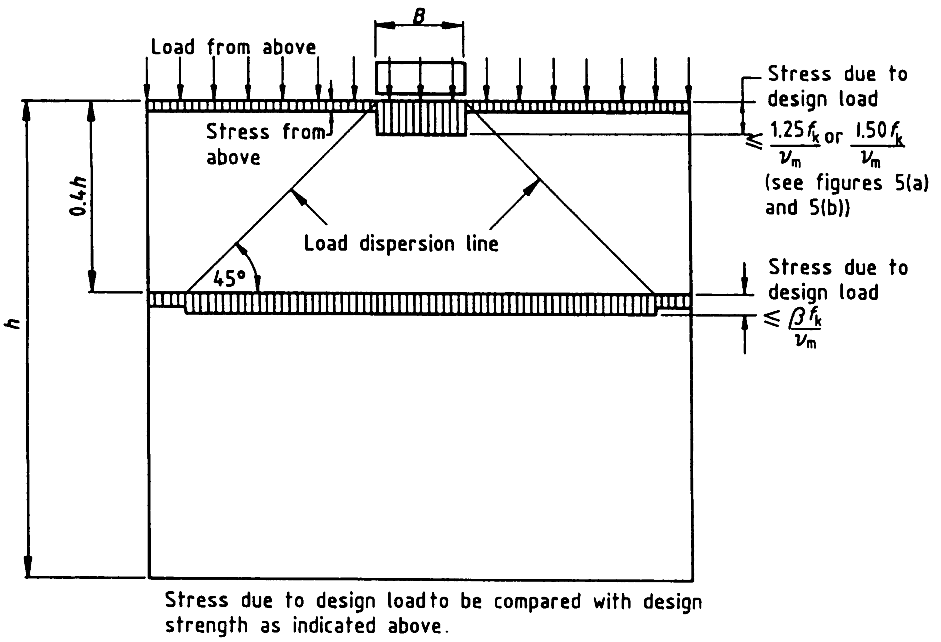
(b) Bearing type 2

Figure 4 — Concentrated loads: types of bearing (*continued*)

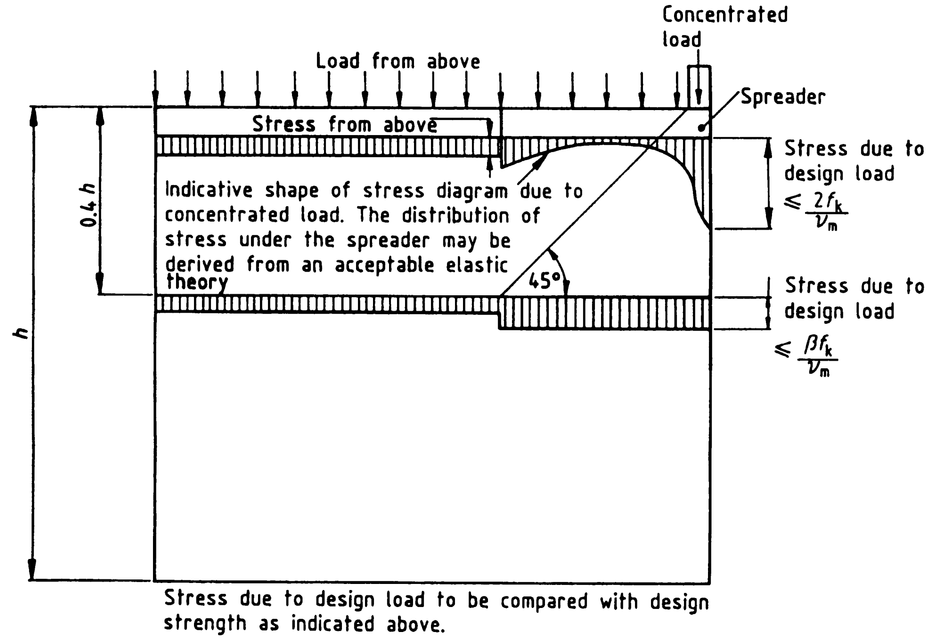


(c) Bearing type 3

Figure 4 — Concentrated loads: types of bearing (*concluded*)



(a) Load distribution for bearing types 1 and 2



(b) Load distribution for bearing type 3

Figure 5 — Concentrated loads: load distribution

31 Composite action between walls and their supporting beams

Where a wall and the beam on which it is supported are designed to act as a single composite unit, the magnitude of the stress concentrations likely to occur near the base of the wall in the vicinity of the beam supports should be assessed. At these positions, the wall should be designed in accordance with the requirements given in Clause **28**, using Clause **30** for concentrated loads and ignoring the effects of slenderness. The remainder of the wall should be designed in accordance with Clause **28**.

32 Walls subjected to lateral load

32.1 General

Empirical guidance on certain wall shapes and conditions is given in BS 5628-3 and in building regulations. This clause enables these walls to be designed by calculation.

The loads to be used in design are given in Clause **18**.

NOTE The presence of a damp-proof course may significantly affect the capacity of masonry to resist flexural tensile stresses at the level of the damp-proof course. See BS 8215:1998, **5.6**.

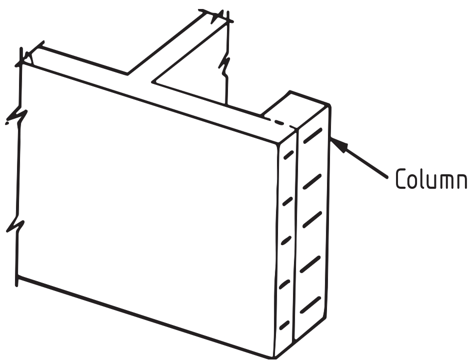
32.2 Supporting conditions and continuity

In assessing the lateral resistance of masonry panels, it is essential that support conditions and continuity over supports are taken into account. Typical examples are illustrated in Figure 6 and Figure 7.

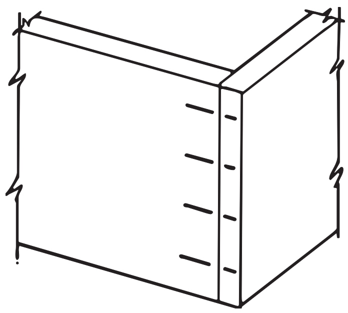
Simple supports may be provided by wall ties conforming to BS EN 845-1, which have a design resistance that exceeds the design loads.

The reaction along an edge of a wall due to the design load may normally be assumed to be uniformly distributed when designing the means of support. The connection to the support may be in the form of wall ties or by the shear resistance of the masonry taking into account the damp proof course, if any.

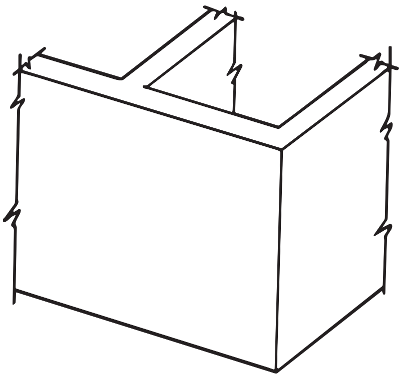
In the case of cavity construction, continuity may be assumed even if only one leaf is continuously bonded over or past a support, provided that the cavity wall has wall ties in accordance with **25.2.4**. Where the leaves are of different thicknesses the thicker leaf is to be the continuous leaf. The load to be transmitted from a panel to its support may be taken by wall ties to one leaf only, provided that there is adequate connection between the two leaves, particularly at the edges of the panels.



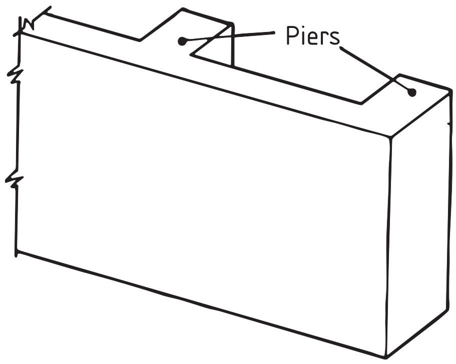
a) Metal ties to columns. Simple support



c) Metal ties to columns or unbonded return walls. Shear and possible moment restraint

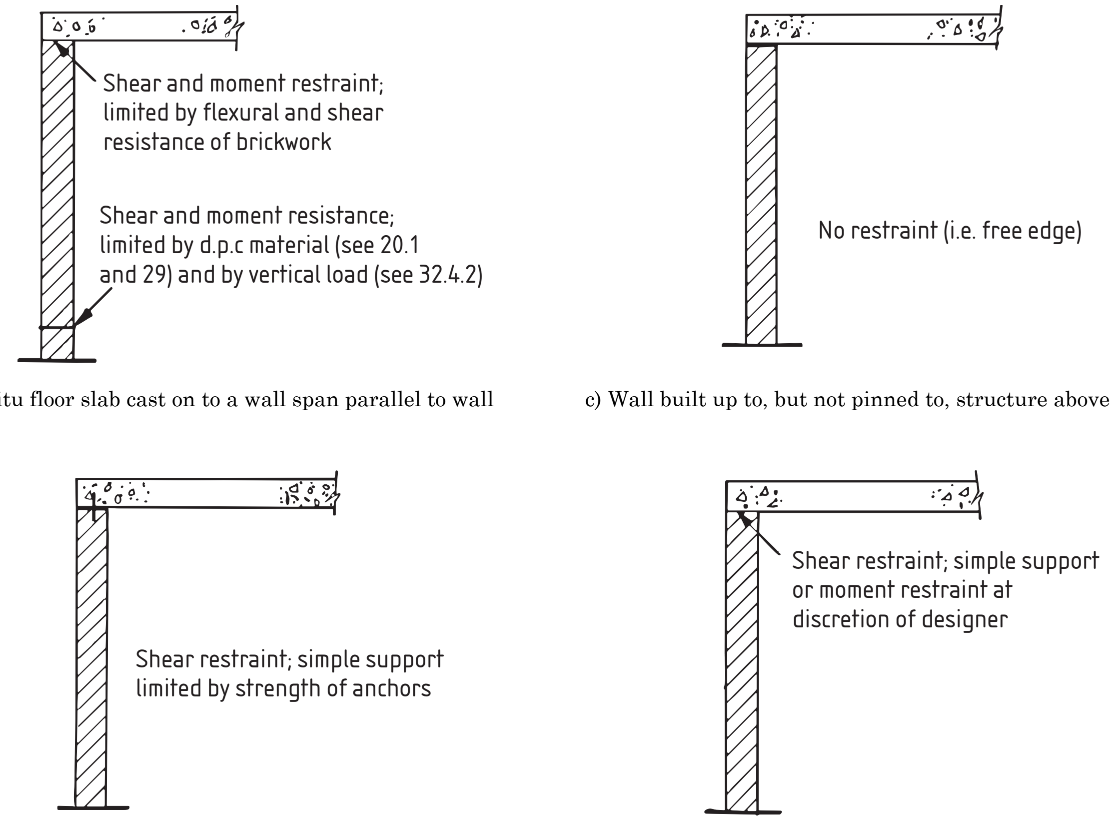


b) Bonded return walls. Restrained support; direct force and moment restraint limited by tensile strength of masonry as given in **20.1**



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

Figure 6 — Vertical support conditions for laterally loaded panels



a) In-situ floor slab cast on to a wall span parallel to wall

c) Wall built up to, but not pinned to, structure above

b) 1) Precast units spanning parallel to wall

2) Walls solidly pinned up to structure above

d) Similar to c) above with suitable anchors

Figure 7 — Horizontal support conditions

32.3 Limiting dimensions

In a laterally-loaded panel or free-standing wall built of masonry set in mortar strength classes M12, M6, M4 or M2 / designations (i), (ii), (iii) and (iv), and designed in accordance with Clause **32**, the dimensions should be limited as follows:

a) Panel supported on three edges;

1) two or more sides continuous: height × length equal to 1 500t_{ef}^2 or less;

2) all other cases: height × length equal to 1 350t_{ef}^2 or less;

b) Panel supported on four edges;

1) three or more sides continuous: height × length equal to 2 250t_{ef}^2 or less;

2) all other cases: height × length equal to 2 025t_{ef}^2 or less;

c) Panel simply supported at top and bottom;

Height equal to 40t_{ef} or less;

d) Free-standing wall;

Height equal to 12t_{ef} or less.

In cases a) and b) no dimension should exceed 50 times the effective thickness t_{ef}.

32.4 Methods of design for laterally loaded wall panels

32.4.1 *General*

Masonry walls subjected to mainly lateral loads are not capable of precise design. There are, however, two approximate methods that at present may be used for assessing the strengths of such walls:

a) as a panel supported on a number of sides;

b) as an arch spanning between suitable supports.

When a wall has openings in it or is of an irregular shape such that this clause cannot be used directly, some guidance is given in Annex E.

32.4.2 *Calculation of design moments in panels*

Masonry walls are not isotropic and there is an orthogonal strength ratio, μ (see **3.19**) depending on the brick or block and mortar used, as may be found from the characteristic flexural strengths given in Clause **20**.

The calculation of the design moment of a panel has to take into account the masonry properties referred to above and may be taken as either:

$\alpha W_k \gamma_f L^2$ per unit height, when the plane of failure (see Table 3) is perpendicular to the bed joints; or,

$\mu \alpha W_k \gamma_f L^2$ per unit length, when the plane of failure (see Table 3) is parallel to the bed joints.

where

α	is the bending moment coefficient taken from Table 8;
γ_f	is the partial safety factor for loads (Clause **18**);
μ	is the orthogonal ratio;
L	is the length of the panel;
W_k	is the characteristic wind load per unit area.

When a vertical load acts so as to increase the flexural strength in the parallel direction, the orthogonal strength ratio μ may be modified by using a flexural strength in the parallel direction of:

$f_{kx} + \gamma_m g_d$

where

f_{kx}	is the flexural strength in the parallel direction, taken from Table 3;
γ_m	is the appropriate partial safety factor for materials (Clause **23**);
g_d	is the design vertical dead load per unit area.

The bending moment coefficient, α, at a damp proof course may be taken as for an edge over which full continuity exists when there is sufficient vertical load on the damp proof course to ensure that its flexural strength (see **19.1**) is not exceeded.

Table 8 gives values of bending moment coefficients, α, for various values of μ, the orthogonal ratio derived from Table 3, modified as necessary for vertical load.

For walls spanning vertically, the design moment per unit length of wall at mid-height of the panel may be taken as:

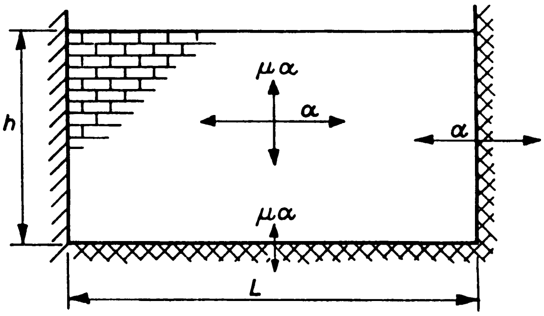
$$\frac{W_k \gamma_f h^2}{8}$$

unless the end conditions justify treating the panel as partially fixed. Piers should be treated in the same way, and the proportion of load being carried by the pier should be assessed from normal structural principles.

Table 8 — Bending moment coefficients in laterally loaded wall panels

NOTE 1 Linear interpolation of μ and h/L is permitted.

NOTE 2 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 less than 0.3 will tend to act as a freestanding wall, whilst the same panel having h/L greater than 1.75 will tend to span horizontally.



Key to support conditions

denotes free edge

simply supported edge

an edge over which full continuity exists

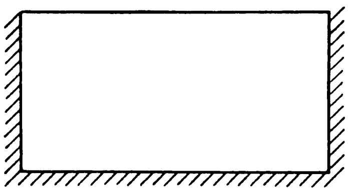
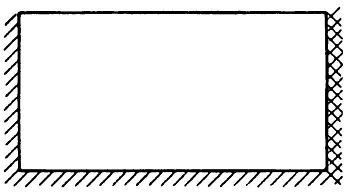
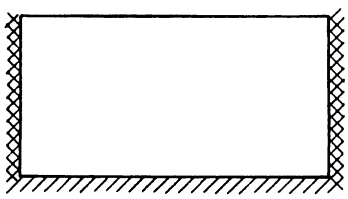
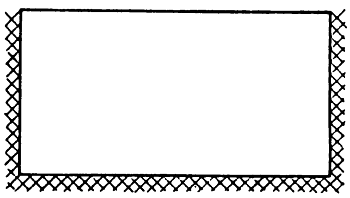
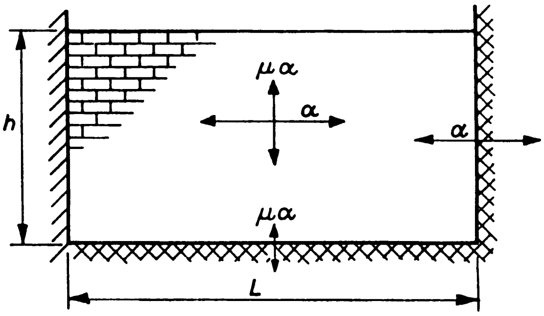
	μ	Values of α						
		h/L						
		0.3	**0.5**	**0.75**	**1.00**	**1.25**	**1.5**	**1.75**
 A	1.00	0.031	0.045	0.059	0.071	0.079	0.085	0.090
	0.90	0.032	0.047	0.061	0.073	0.081	0.087	0.092
	0.80	0.034	0.049	0.064	0.075	0.083	0.089	0.093
	0.70	0.035	0.051	0.066	0.077	0.085	0.091	0.095
	0.60	0.038	0.053	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 8 — Bending moment coefficients in laterally loaded wall panels (*continued*)

NOTE 1 Linear interpolation of μ and h/L is permitted.

NOTE 2 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 less than 0.3 will tend to act as a freestanding wall, whilst the same panel having h/L greater than 1.75 will tend to span horizontally.

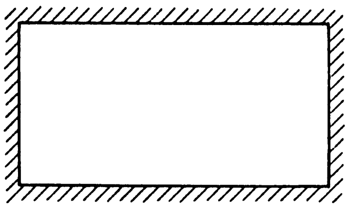


Key to support conditions

denotes free edge

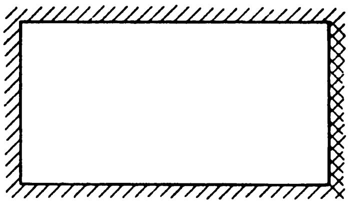
simply supported edge

an edge over which full continuity exists



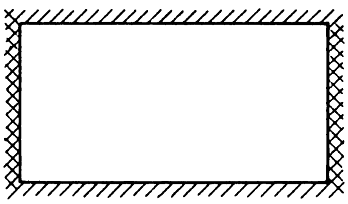
E

μ	Values of α, h/L = 0.3	0.5	0.75	1.00	1.25	1.5	1.75
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



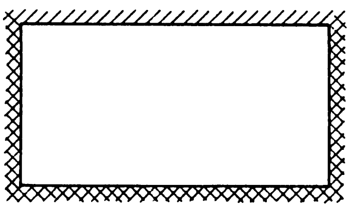
F

μ	Values of α, h/L = 0.3	0.5	0.75	1.00	1.25	1.5	1.75
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

μ	Values of α, h/L = 0.3	0.5	0.75	1.00	1.25	1.5	1.75
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

μ	Values of α, h/L = 0.3	0.5	0.75	1.00	1.25	1.5	1.75
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

Table 8 — Bending moment coefficients in laterally loaded wall panels (*continued*)

NOTE 1 Linear interpolation of μ and h/L is permitted.

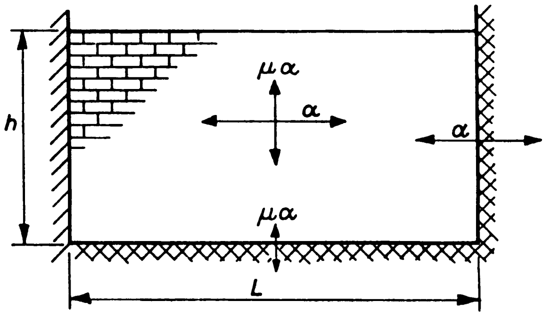
NOTE 2 When the dimensions of a wall are outside the range of h/L given in this table, it will usually be sufficient to calculate the moments on the basis of a simple span. For example, a panel of type A having h/L less than 0.3 will tend to act as a freestanding wall, whilst the same panel having h/L greater than 1.75 will tend to span horizontally.

Key to support conditions

denotes free edge

simply supported edge

an edge over which full continuity exists



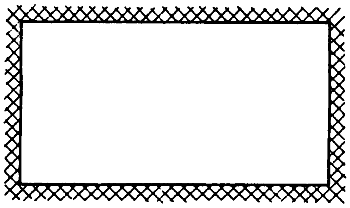
	μ	**Values of α**						
		h/L						
		0.3	**0.5**	**0.75**	**1.00**	**1.25**	**1.5**	**1.75**
 I	1.00	0.004	0.009	0.015	0.021	0.026	0.030	0.033
	0.90	0.004	0.010	0.016	0.022	0.027	0.031	0.034
	0.80	0.005	0.010	0.017	0.023	0.028	0.032	0.035
	0.70	0.005	0.011	0.019	0.025	0.030	0.033	0.037
	0.60	0.006	0.013	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 8 — Bending moment coefficients in laterally loaded wall panels (*concluded*)

NOTE 1 Linear interpolation of μ and h/L is permitted.

NOTE 2 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 less than 0.3 will tend to act as a freestanding wall, whilst the same panel having h/L greater than 1.75 will tend to span horizontally.

Key to support conditions

denotes free edge

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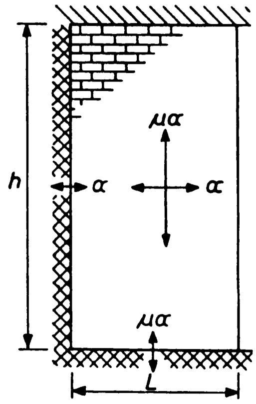
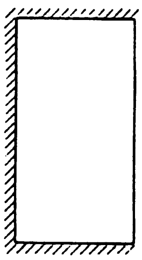
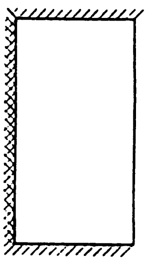
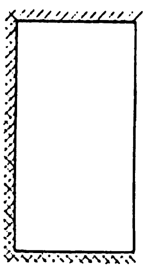
an edge over which full continuity exists



	μ	Values of α						
		h/L						
		0.3	**0.5**	**0.75**	**1.00**	**1.25**	**1.5**	**1.75**
 **J**	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
 **K**	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
 **L**	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

32.4.3 ***Calculation of design moment of resistance of panels***

The design moment of resistance of a masonry wall is given by:

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

where

f_{kx}	is the characteristic flexural strength appropriate to the plane of bending (Clause **21**);
γ_m	is the appropriate partial safety factor for materials (see Table 4);
Z	is the section modulus.

In assessing the section modulus of a wall including piers, the outstanding length of flange from the face of the pier should be taken as:

a) 4 × thickness of wall forming the flange when the flange is unrestrained; or

b) 6 × thickness of wall forming the flange when the flange is continuous;

but in no case more than half the clear distance between piers.

32.4.4 ***Arching***

When a masonry wall is built solidly between supports capable of resisting an arch thrust (see below) or when a number of walls are built continuously past supports, the wall may be designed assuming that a horizontal arch develops within the thickness of the wall. A method of designing such a wall is given below. In the present state of knowledge, walls subjected to mainly lateral loads should be designed only for arching horizontally, but a method for designing walls arching vertically under axial load is given in **32.8**.

Calculation should be based on a simple three-pin arch and the bearing at the supports and at the central hinge should be assumed as 0.1 times the thickness of the wall.

For walls having a length to thickness ratio of 25 or less, the deflection under the design lateral load can be ignored. For other walls, allowance for the deflection should be made.

The arch rise is given by:

$$t - \frac{t}{10} - \delta$$

where

t	is the overall thickness of the wall;
δ	is the deflection under the design lateral load ($\delta = 0$ for walls of length/thickness less than 25).

The arch thrust has to be assessed from knowledge of the applied lateral load, the strength of the masonry in compression and the effectiveness of the junction between the wall and the support resisting the thrust. A small change in length of a wall in arching can considerably reduce the arching resistance; therefore care should be taken if the masonry is built of masonry units that may shrink in service. Provided that the junction between the masonry and the support is solidly filled with mortar, the maximum design arch thrust per unit width of wall may be assumed to be:

$$1.5 \frac{f_k}{\gamma_m} \left(\frac{t}{10} \right)$$

For cases where the lateral deflection will be small, and hence can be ignored, the design lateral strength is given by:

$$q_{lat} = \frac{f_k}{\gamma_m}\left(\frac{t}{L}\right)^2$$

where

q_{lat}	is the design lateral strength per unit area of wall;
t	is the overall thickness of wall;
f_k	is the characteristic compressive strength of the masonry;
L	is the length of the wall;
γ_m	is the appropriate partial safety factor for materials (see Table 4).

The supporting structure has to be designed to be capable of resisting the arch thrust with negligible deformation.

32.4.5 *Design lateral strength for cavity walls*

The design lateral strength for a cavity wall tied with wall ties capable of transmitting the tensile and compressive forces to which they are subjected, may be taken as the sum of the design lateral strengths of the two leaves, allowing for the additional strength of any piers bonded to one or both of the leaves.

Where the wall ties are not capable of transmitting the full force, the contribution of the appropriate leaf should be limited accordingly.

32.5 Method of design for free-standing walls

32.5.1 *General*

Free-standing walls should be designed as cantilevers springing from the top of the foundation or from the point of horizontal lateral restraint when such restraint is sufficient to resist the horizontal reaction from the wall, except that wall panels between piers may be designed as three-sided or horizontally spanning in accordance with **32.4.3** and **32.4.4** respectively. The piers should then be designed as cantilevers to resist the reaction from the panel. Mortar should not be weaker than strength class M4 (see Table 1).

32.5.2 *Calculation of design moment in free-standing walls*

The design moment of a free-standing wall subjected to horizontal forces is given by:

$$W_k \gamma_f \frac{h^2}{2} + Q_k \gamma_f h_L$$

where

W_k	is the characteristic wind load per unit area (see Clause **17**);
γ_f	is the partial safety factor for loads (see Clause **18**);
h	is the clear height of the wall or pier above restraint;
Q_k	is the characteristic imposed load (see Clause **17**);
h_L	is the vertical distance between the point of application of the horizontal load, Q_k, and the lateral restraint.

32.5.3 ***Calculation of design moment of resistance of free-standing walls***

The design moment of resistance across the bed joints is given by:

$$\left(\frac{f_{kx}}{\gamma_m} + g_d\right) Z$$

where

f_{kx}	is the characteristic flexural strength at the critical section, which may be the damp proof course (Clause **20**);
γ_m	is the appropriate partial safety factor for materials (Table 4);
Z	is the section modulus, which may take into account any variation on the plan arrangement, e.g. chevron, curved or zig-zag walls (in the case of walls with piers, see **32.4.3**);
g_d	is the design vertical dead load per unit area.

In cases where the flexural strength of the masonry cannot be relied upon (see **20.1**) a free-standing wall can only be used when there is sufficient vertical load acting. The design moment of resistance per unit length may then be assessed by assuming that the vertical load is resisted by a rectangular stress block when the moment in a wall of uniform thickness is:

$$\frac{n_w}{2}\left[t - \frac{n_w \gamma_m}{f_k}\right]$$

where

t	is the thickness of the wall;
n_w	is the design vertical load per unit length of wall;
f_k	is the characteristic compressive strength of masonry (Clause **19**);
γ_m	is the appropriate partial safety factor for materials (Table 4).

32.6 Retaining walls

Where retaining walls are to resist lateral earth and water pressure only, they should be designed in accordance with Clauses **17** and **18**.

32.7 Foundation walls

Where a wall carries vertical loading combined with lateral loading the earth and water pressure should be treated as detailed in **32.6**.

32.8 Design lateral strength of axially loaded walls and columns

The design lateral strength of axially loaded walls and columns may be calculated from consideration of the effective eccentricity due to the lateral load and any other eccentricity, using Clause **28** and Annex B, or from the relationship:

$$q_{\mathrm{lat}} = \frac{4tn}{h_{\mathrm{a}}^{2}}$$

NOTE The formula incorporates a safety factor of 2.

where

q_{lat}	is the design lateral strength per unit area of wall or column;
n	is the axial load per unit length of wall available to resist the arch thrust; for normal design it should be based on the characteristic dead load, but when considering the possible effects of misuse or accident, n should be the appropriate design load (see Clause **18**);
h_{a}	is the clear height of the wall between concrete surfaces or other construction capable of providing adequate resistance to rotation across the full thickness of a wall;
t	is the actual thickness of wall or column;

provided that:

— the wall or column is contained between concrete floors or other construction affording adequate lateral support and adequate resistance to rotation across the full width of the member; and,

— any damp proof course or other plane of low frictional resistance in the wall or column can transmit the relevant horizontal forces; and,

— the design load is not less than 0.1 N/mm^2; and,

— the ratio h_{a}/t does not exceed 25 in the case of narrow brick walls or 20 for all other types of wall.

If the wall or column is supported by returns which are capable of resisting the horizontal reaction transmitted to them, on a vertical edge or edges, the value of q_{lat} as given by the above equation may be multiplied by the factor k from Table 9 in which h_{a} is the clear height of the wall or column, and L is the length of the wall or column.

Table 9 — Factor *k*

Number of returns	Value of *k*				
	L/h_{a}	0.75	1.0	2.0	3.0
1		1.6	1.5	1.1	1.0
2		4.0	3.0	1.5	1.2

32.9 Method of design for propped cantilever walls for single storey buildings under wind loading

32.9.1 ***General***

Propped cantilever walls for single storey buildings under wind loading should be designed as springing from the top of the foundation or from the point of horizontal lateral support when such support is sufficient to resist the horizontal reaction from the wall.

The lateral support (prop) at the top of the wall, which may be provided by a braced roof system, should be capable of resisting the lateral forces. To accommodate a redistribution of moment resulting from possible deflections of the upper support, the design moment of resistance at the lower support should be generated from gravity forces only, i.e. by assuming a cracked section. No flexural tensile strength should be taken into account at this level under any condition.

Under the ultimate limit state the member should be designed to satisfy all the following conditions.

a) The design flexural tensile stress in the wall (other than in the vicinity of the lower support) should not exceed the appropriate value of design flexural strength (Clauses **20** and **23**). At the lower support, the design moment of resistance due to gravity forces should be assessed using factors of safety of 0.9 on the dead load and 1.4 on the wind load.

b) The wall should be stable when its strength is derived from gravity forces only (i.e. when the flexural tensile strength of the masonry is ignored) with factors of safety of 1.0 on the dead load and on the wind load.

c) The design stress in the compressive zone of the wall should not exceed the design compressive strength.

32.9.2 ***Calculation of design moment***

For the ultimate limit state, the maximum design moment at any section should be calculated by statics superimposing the gravity moment on the moment resulting from lateral loads, assuming the wall to be simply supported at the top and the base. Characteristic loads given in Clause **17** and relevant partial safety factors for loads given in Clause **18** should be used.

In calculating the design moment, the height of the wall should be taken as the distance between the upper and lower lateral supports.

32.9.3 ***Calculations of design moment of resistance***

32.9.3.1 *Condition a) of **32.9.1***

For this condition the design moment of resistance across the bed joints for use other than in the vicinity of the lower lateral support is given by:

$$\left(\frac{f_{kx}}{\gamma_m} + g_d\right)Z$$

where

f_{kx}	is the characteristic flexural strength;
γ_m	is the appropriate partial safety factor for materials (see Table 4);
Z	is the section modulus relevant to the plan arrangement, e.g. solid, curved, piered or diaphragm forms;
g_d	is the design vertical dead load per unit area.

32.9.3.2 *Condition b) of **32.9.1***

For this condition the design moment of resistance across any bed joint is given by:

$n_g x$

where

x	is the internal lever arm of the resisting moment taken on the appropriate side of the section;
n_g	is the design vertical load at the section under consideration.

NOTE In calculating n_g account should be taken of any uplift forces due to wind action on the roof of the building.

32.9.4 *Integrity of sections in shear*

32.9.4.1 *Vertical direction*

The vertical shear stress between two elements of a section, such as the junction of the outer or inner leaf and the cross rib of a diaphragm wall should be resisted by one of the following.

a) Bonded masonry where the vertical shear stress induced by bending is resisted by masonry units bridging the interface between the two elements of the section. The design vertical shear stress due to lateral loads should be less than the design shear strength derived from Clause **21**.

b) Shear ties conforming to BS EN 845-1 spaced at centres not exceeding 450 mm vertically and selected to provide a design vertical shear resistance (see **24.2.2**) which exceeds the design vertical shear load per tie.

c) Appropriate flat metal sections in the bed joints acting as shear connectors, the size and spacing of which should be calculated as follows:

$$ru = \frac{12 t_w s v}{(0.87 f_{yt})}$$

where

r	is the width of the connector;
u	is the thickness of the connector;
t	is the width of the masonry section in vertical shear;
s	is the spacing of the connectors;
v	is the design vertical shear stress on the masonry section;
f_{yt}	is the declared tensile strength of the connector (see **C.5**).

The flat metal sections should also conform to the recommendations for wall ties in respect of anchorage and embedment depth.

32.9.4.2 *Horizontal direction*

The design shear stress in the horizontal direction calculated on that part of the section which resists shear force should not exceed the value of the design shear strength for the horizontal direction in the horizontal plane derived from Clause **21**.

Section 5. Design: accidental damage

33 Design: accidental damage

33.1 General guidance

The general recommendations made in **16.3** are aimed at the limitation of accidental damage and preservation of structural integrity and are applicable to all building classes.

No additional detailed recommendations are made in respect of Class 1 buildings.

For Class 2A buildings, effective anchorage of suspended floors to walls, or the provision of effective horizontal ties, is recommended.

For Class 2B buildings, horizontal tying requirements and recommendations for the provision within the structure of vertical tying are given, as is assessment of residual stability and spread of damage, following the removal of a loadbearing element, as defined in Table 10.

Class 3 buildings are not fully covered by this British Standard but some aspects are.

The recommendations are listed in Table 11.

If option (1) in Table 11 is adopted, the structure should be examined for the effect of the removal, within each storey, of each supporting column, and each beam supporting one or more columns or a loadbearing wall, or any nominal length of loadbearing wall, one at a time, unless such columns, beams or loadbearing walls are designed as key elements. In the case of option (2) no further assessment of residual stability or spread of damage will normally be required.

The designer should satisfy himself that the damaged structure has adequate residual stability and that collapse of any significant portion of the structure is unlikely to occur.

33.2 Key element

A key element is a member which, together with its essential supports, can withstand, without collapse, its reduced design load in accordance with Clause **18**d), and an accidental design load of 34 kN/m^2. This accidental load shall be applied from any direction together with the reaction, if any, which could be expected to be directly transmitted to that member by any attached building component also subjected to the load of 34 kN/m^2 from the same direction. The reaction from the attached member may be reduced with regard to the strength of the attached component and the strength of its connection.

A masonry column or wall may have adequate strength to withstand a lateral design pressure of 34 kN/m^2 if it supports a sufficiently high vertical axial load. The lateral strength of masonry can be checked in accordance with **32.8** but using the following formula:

$$q_{\mathrm{lat}} = \frac{7.6tn}{h_{\mathrm{a}}^{2}}$$

NOTE The formula incorporates a safety factor of 1.05.

Table 10 — Loadbearing elements

Type of loadbearing element	Extent
Beam or slab supporting one or more columns or a loadbearing wall	Clear span between supports or between a support and the extremity of a member
Column	Clear height between horizontal lateral supports
Wall incorporating one or more lateral supports (see Note 2)	Length between vertical lateral supports or length between a vertical lateral support and the end of the wall
Wall without lateral supports	Length not exceeding 2.25h anywhere along the wall (for internal walls) Full length (for external walls)

NOTE 1 Under accidental loading conditions, temporary supports to slabs can be provided by substantial or other adequate partitions capable of carrying the required load.

NOTE 2 Lateral supports to walls can be provided by intersecting or return walls, piers, stiffened sections of wall, substantial non-loadbearing partitions in accordance with **33.6**a), b) and c) or purpose-designed structural elements.

33.3 Partial safety factors

For consideration of misuse and accident, the partial safety factors for loads should be taken from Clause **18**d) and those for material strength γ_m from **23.3**, **23.4**, **23.5** and **33.2**.

33.4 Horizontal ties

The requirements for peripheral, internal and column or wall ties can be obtained from Table 12.

Peripheral, internal and column wall ties should be provided at each floor level and at roof level, but where the roof is of lightweight[1)] construction no such ties need be provided at that level.

Horizontal ties may be provided in whole or in part by structural members that may already be stressed to their design values in serving other purposes, e.g. reinforcement in a concrete floor slab or masonry in tension. Ties should be positioned to resist accidental damage most effectively. In the case of Class 2A buildings, the horizontal tying action may be provided by effectively anchoring floors of in-situ or precast concrete, or timber floor joists, to the masonry walls in accordance with Annex D or BS 8103-1.

33.5 Vertical ties

The requirements for full vertical ties can be obtained from Table 13.

For full vertical tying (option (2) in Table 11), precast or in-situ concrete or other heavy floor or roof units should be anchored, in the direction of their span, either to each other over a support or directly to their supports, in such a manner as to be capable of resisting a horizontal tensile force of F_t kN/m width, where F_t is as given in Table 12. The wall should be contained between concrete surfaces or other similar construction, excluding timber, capable of providing resistance to lateral movement and rotation across the full width of the wall.

Vertical ties should extend from roof level to the foundation or to a level at and below which the relevant members of the structure are protected in accordance with **33.2**.

They should be fully anchored at each end and at each floor level and any joint should be capable of transmitting the required tensile forces.

Ties should be adequately safeguarded from damage and corrosion.

1) Roofs comprising timber or steel trusses, flat timber roofs or roofs incorporating concrete or steel purlins with mineral fibre cement sheets or wood-wool deck may be regarded as lightweight.

33.6 Loadbearing elements

For the purposes of this clause the definition of "loadbearing elements" is as given in Table 10, where *h* is the clear height of wall between horizontal lateral supports.

Lateral supports in relation to wall elements as defined in Table 10 may be considered to be provided by the following:

a) an intersecting or return wall tied to a wall to which it affords support, with connections capable of resisting a force of F_t kN/m height of wall (Table 12), having a length without openings of not less than $h/2$ at right angles to the wall afforded support and having an average weight of not less than 340 kg/m^2;

b) a pier or a stiffened section of the wall (not exceeding 1 m in length), capable of resisting a horizontal force of 1.5 F_t kN/m height of wall (Table 12);

c) a substantial partition at right angles to the wall having an average weight of not less than 150 kg/m^2, tied with connections capable of resisting a force of 0.5 F_t kN/m height of wall (Table 12).

NOTE A substantial partition need not be in a straight line but should in effect divide the bay into two compartments.

Table 11 — Detailed accidental damage recommendations

<table>
<tr><th>Building Class</th><th colspan="2">Recommendations</th></tr>
<tr><td>Class 1</td><td colspan="2">Provide robustness, interaction of components and containment of spread of damage (see Clause 16).</td></tr>
<tr><td>Class 2A</td><td colspan="2">As for Class 1, and additionally provide effective anchorage of all suspended floors to walls or effective horizontal ties in accordance with 33.4 and Table 12.</td></tr>
<tr><td rowspan="2">Class 2B</td><td colspan="2">As for Class 1, and additionally:</td></tr>
<tr><td><u>Option (1)</u>
Provide (other than key elements), supporting columns, beams or slabs supporting one or more columns or a loadbearing wall, or loadbearing walls removable, one at a time, without causing collapse.</td><td><u>Option (2)</u>
Provide effective horizontal ties in accordance with 33.3 and Table 13, and vertical ties in accordance with 33.5 and Table 12.</td></tr>
</table>

Table 12 — Requirements for full peripheral, internal and column or wall ties

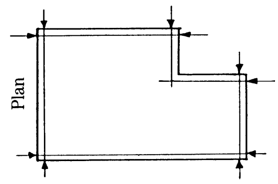
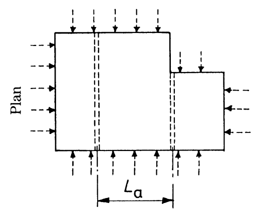
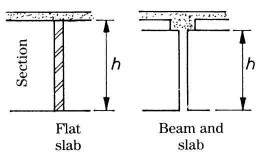
Basic horizontal tie force = F_t = 60 kN or 20 + 4N_S kN, whichever is the lesser of the two values; N_S is the number of storeys (including ground and basement).				
Type of tie	**Unit of tie force**	**Size of design tie force**	**Location of tie force (arrowed)**	**Fixing requirements and notes**
A. Peripheral	kN	F_t	Around whole perimeter 	Ties should be: a) placed within 1.2 m of edge of floor or roof or in perimeter wall; b) anchored at re-entrant corners or changes of construction.
B. Internal (both ways)	kN/m widt	F_t or $\frac{F_t(G_k + Q_k)}{7.5} \times \frac{L_a}{5}$ F_t F_t or $\frac{F_t(G_k + Q_k)}{7.5} \times \frac{L_a}{5}$ whichever is the greater	***One way spans*** (i.e. in cross wall or spine construction) i) in direction of span ii) in direction perpendicular to span ***Two way spans*** (in both directions)  	a) Internal ties should be anchored to perimeter ties or continue as wall or column ties. b) Internal ties should be provided: 1) uniformly throughout floor or roof width; or, 2) concentrated (6 m max. horizontal tie spacing); or, 3) within walls 0.5 m max. above or below the floor or roof and at 6 m max. horizontal spacing; 4) in addition to peripheral ties spaced evenly in perimeter zone. c) Calculation of tie forces should assume: 1) (G_k + Q_k) as the sum of average characteristic dead and imposed loads in kN/m^2; 2) L_a as the lesser of: the greatest distance in metres in the direction of the tie, between the centres of columns or other vertical loadbearing members whether this distance is spanned by a single slab or by a system of beams and slabs; or 5 × clear storey height h.

Table 12 — Requirements for full peripheral, internal and column or wall ties (*concluded*)

Basic horizontal tie force = F_t = 60 kN or 20 + 4N_S kN, whichever is the less of the two values; N_S is the number of storeys (including ground and basement).

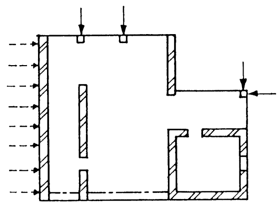
Type of tie	Unit of tie force	Size of design tie force	Location of tie force (arrowed)	Fixing requirements and notes
C. External column	kN	2F_t or (h/2.5)F_t, whichever is the lesser, where h is in metres	 Plan	a) Corner columns should be tied in both directions. b) Tie connection to masonry may be based on shear strength or friction (but not both). c) Wall ties (where required) should be: 1) spaced uniformly along the length of the wall; 2) concentrated at centres not more than 5 m apart and not more than 2.5 m from the end of the wall. d) External column and wall ties may be provided partly or wholly by the same reinforcement as perimeter and internal ties.
D. External wall	kN/m length of loadbearing wall			

Table 13 — Requirements for full vertical ties

Minimum thickness of a solid wall or one loadbearing leaf of a cavity wall	140 mm
Minimum characteristic compressive strength of masonry	5 N/mm^2
Maximum ratio h_a/t	20
Allowable mortar strength classes / designations (see Table 1)	M4 (iii), M6 (ii), M12 (i)
Tie force	$\frac{34A}{8\,000}\left(\frac{h_a}{t}\right)^2$ N or 100 kN/m length of wall or per column, whichever is the greater
Positioning of ties	5 m centres max. along the wall and 2.5 m max. from an unrestrained end of any wall

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

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

t is the thickness of column or wall.

Annex A (normative) Mortar testing of site made mortar

A.1 Preliminary tests

At least six weeks before the masonry building that will use site made mortar is started, the strength of the mortar designations proposed for use should be determined in the laboratory, with materials from the sources from which the site is to be supplied. Six 100 mm × 40 mm × 40 mm prisms made with mortar of a consistency corresponding to that required on site should be made, cured hydraulically and tested for compression strength, in accordance with the procedures given in BS EN 1015-11.

A.2 Interpretation of test results

The compressive strength to be expected from the various mortar designations is shown in Table 1. If desired, half of the test specimens may be tested at seven days. Normally, the results of these tests will give an indication of the strength to be expected at 28 days. For mortars included in Table 1, the strength at 7 days will approximate to two-thirds of the strength at 28 days provided that the mortars are based on Portland cement without any additives to retard or accelerate the rate of hardening. If the average of these 7-day strengths equals or exceeds two-thirds of the appropriate strength given in Table 1, the mortar requirements are likely to be satisfied.

If less, then the designer may choose to await the 28-day strength or have the tests repeated using a more suitable sand. If the 28-day figure fails to achieve the strength given in Table 1, either the tests should be repeated, using a more suitable sand, or the next higher designation of mortar should be used. If the latter procedure is adopted, the strength required of this higher designation should not be that given in Table 1, but should be the strength corresponding to the designation first chosen.

A.3 Site tests

Six 100 mm × 40 mm × 40 mm prisms should be prepared on site for every 150 m^2 of wall, using any one designation of mortar, or for every storey of the building, whichever is the more frequent. Specimens should be stored and tested in accordance with BS EN 1015-11. Half of the site samples should be tested at 7 days. The average strength should exceed two-thirds of the appropriate 28-day strength given in Table 1.

When the remaining site samples are tested at the age of 28 days, the mortar will be deemed to pass if the average of the values obtained from three prisms or the average of the values obtained from three cubes exceeds the appropriate site values given in Table 1.

Mortar should be sampled in accordance with BS EN 1015-2.

Annex B (normative) Derivation of β

B.1 Assumptions for eccentricity and slenderness

The eccentricity is assumed to vary from the value e_x at the top of the wall, calculated in accordance with Clause **30**, to zero at the bottom of the wall, subject to an additional eccentricity being considered to cover slenderness effects. No slenderness effect need be considered for walls or columns where the slenderness ratio is less than or equal to 6. The additional eccentricity may be assumed to vary linearly from zero at top and bottom of the wall, to a value e_a over the central fifth of the wall height where e_a is given by:

$$e_a = t\left[\frac{1}{2\,400}(h_{ef}/t_{ef})^2 - 0.015\right] \tag{1}$$

where

- t is the thickness of the wall (or depth of column);
- t_{ef} is the effective thickness of the wall or column;
- h_{ef} is the effective height of the wall or column.

The total design eccentricity, e_t, in the mid-height region of a slender wall is therefore given by:

$$e_t = 0.6\ e_x + e_a \tag{2}$$

where e_x is the eccentricity calculated at the top of the wall.

It should be noted that e_t can be less than e_x and plainly in such cases e_x, the eccentricity at the top of the wall, should govern the design, and should be taken as the design eccentricity.

B.2 Assumption for design of wall made from solid units

For design eccentricities, e_m, of 0 to $0.05t$, calculated from **B.1**, equation (2), the design vertical load capacity of a member is given by:

$$\beta t(f_k/\gamma_m)$$

where

e_m is the larger of e_x and e_t; and $\beta = 1$.

For design eccentricities, e_m, greater than $0.05t$, the eccentric load should be assumed to be resisted by a rectangular stress block with a constant stress of 1.1 f_k/γ_m (see Figure B.1). It follows that the design vertical load capacity of the member is:

$$1.1\left(1 - \frac{2e_m}{t}\right)\left(t \cdot \frac{f_k}{\gamma_m}\right) \tag{3}$$

where

- e_m is the larger of e_x and e_t, but not less than $0.05t$;
- f_k is the characteristic strength of masonry as defined in Clause **22**;
- γ_m is the appropriate partial safety factor for materials as defined in Table 4.

Comparing the expressions for capacity given in **28.2** with equation (3), it will be seen that:

$$\beta = 1.1\ [2\ e_m/t)] \tag{4}$$

The values of β in Table 7 have been calculated from equation (4).

B.3 Alternative assumptions for design of single-leaf walls with hollow concrete blocks

Single-leaf walls with hollow concrete blocks may be designed on the assumption of a stress block acting on the net area using a characteristic strength enhanced in the ratio of gross area to net area. It is conservative, however, to assume for design purposes that the units are solid, using the characteristic strength based on the gross unit area, and this is why no distinction is drawn in Clause **31**.

B.4 Alternative assumptions for design of single-leaf walls of shell bedded blocks or hollow clay masonry with divided bed joints

A stress block approach may also be used for single-leaf walls of shell bedded blocks or hollow clay masonry with divided bed joints as indicated in **B.3** but it is sufficient and conservative to treat them as solid walls, provided the strength is derived as described in Clause **31**.

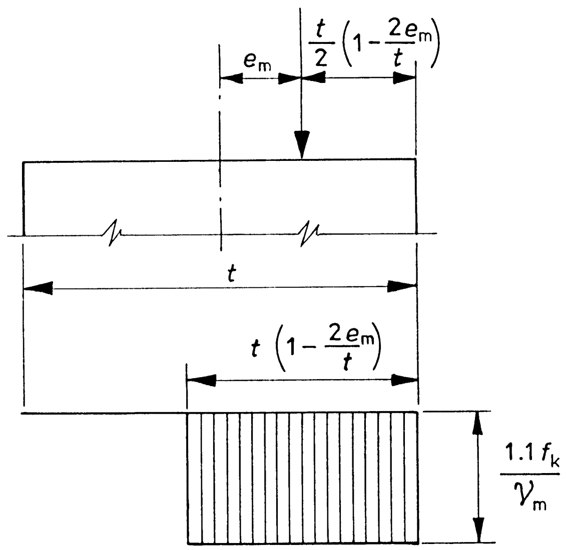


Figure B.1 — Stress block under ultimate conditions

Annex C (informative)
User categories for the selection and application of wall ties

C.1 General

Wall ties are grouped into user categories for convenience of selection and application. Such grouping enables ties, meeting a combination of performance criteria, to be referenced more easily in design guidance, design specifications and execution instructions. The type references and qualifying criteria for the user grouping of wall ties are given in Table C.1, Table C.2 and Table C.1.

C.2 Classification

Ties are classified according to their end use as shown in Table C.1.

Table C.1 — Classification of ties by end use

Classification	Field of use		
	Type of structure	Tie density	Geographical location
Type 1. (Masonry : Heavy duty)	Suitable for most masonry cavity and cladding walls and most building sizes and types. Not very flexible and should not be specified where large adjustments are likely to be needed during construction, where large differential movements are expected to take place between the leaves, or where very low strength/density masonry units are in use.	2.5 ties/m^2 in main areas. 3-4 ties/m run at unbonded edges	Suitable for use on most sites. However, for relatively tall buildings located in the north western fringes of the UK — particularly on coastal sites — and for buildings of unusual shapes, the necessary tie provision should be calculated.
Type 2. (Masonry : General purpose)	Suitable for domestic dwellings and small commercial buildings of a height of up to 15 m above ground level, made with box-form masonry walls comprising two leaves of brickwork or blockwork of similar thickness in the range 90 mm–150 mm. May be suitable for cavity walls having leaves of disparate thickness or stiffness or for cladding walls (having none or limited horizontal spanning capability) and for heights of buildings exceeding 15 m, but should only be used in these situations if shown to be of adequate performance by calculation.	As type 1	Suitable for buildings on flat sites where the basic wind speed is up to 31 m/sec except areas where the site is at an altitude of 150 m or more above sea level. May be adequate for higher altitudes and sloping sites exceeding a slope of 1 in 20 if calculated.
Type 3. (Masonry : Basic)	As type 2.	As type 1	As Type 2 but basic wind speed limited to 25 m/sec.
Type 4. (Masonry : Light duty)	Suitable only for masonry cavity walls, comprising two leaves of similar thickness in the range 90 mm–150 mm, in box-form domestic dwellings of up to 10 m in height. Not suitable for cavity walls having leaves of disparate thickness or stiffness, for cladding walls of any type or for multi-storey structures, of more than three storeys.	As type 1	Suitable for flat sites within towns and cities anywhere in the UK except the north western fringes of Scotland and Ireland (where the basic wind speed exceeds 25 m/sec) and any areas where the site is at an altitude of 150 m or more above sea level.
NOTE There are no limits on the length of ties or technically, on the width of the cavity over which they are designed to function, but the cavity width range for which each product is designed is declared by the manufacturer and the performance data measured for the worst case (usually the widest). See Table C.3.			

C.3 Functional sections

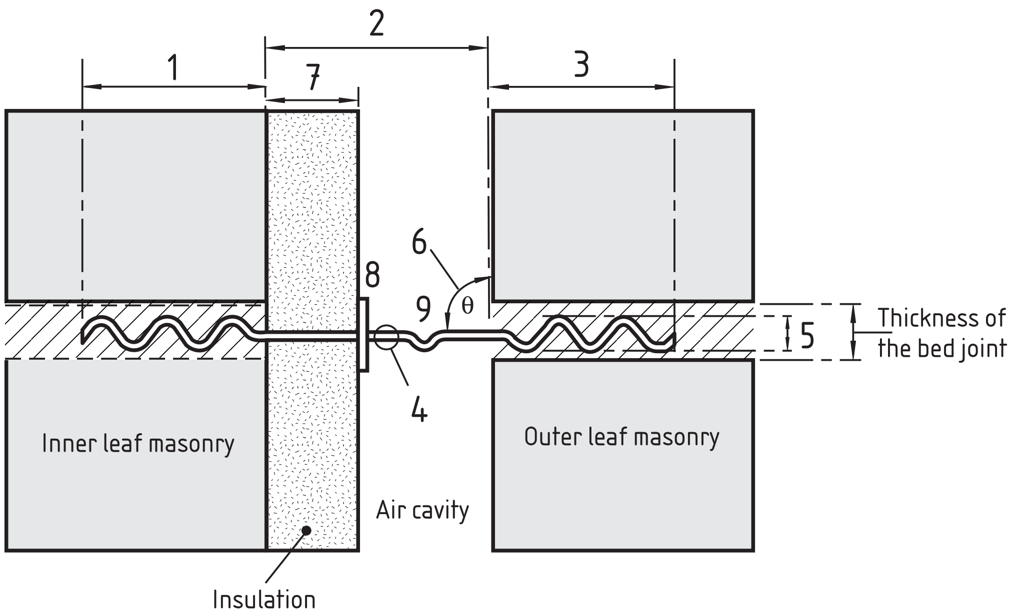
The tie should consist of three functional sections, namely:

a) an inner lead connection;

b) a cavity spanning section;

c) an outer lead connection.

The classification of ties by end use is given in Table C.2 and Figure C.1 for the appropriate type of tie; the angle, θ, should be limited to between 65° and 90°, and drips should be approximately in the centre of the airspace.



Key

1 Embedment length in inner leaf
2 Cavity width
3 Embedment length in outer leaf
4 Diameter or thickness of tie
5 Profile height of the tie
6 The angle between the tie and the wall
7 Thickness of the insulation layer
8 Insulation retention clip
9 Drip to prevent water transmission

Figure C.1 — Definitions for functional sections of masonry–masonry wall ties in an external cavity wall

Table C.2 — Functional sections (see Figure C.1)

Type (see **Table C.1**)	**Inner leaf connection**	**Cavity spanning section**	**Outer leaf connection**
Type 1	Should consist of a rod, wire, bar or plate formed in such a way as to be able to bond to mortar. Should have a thickness of material not exceeding half the design joint thickness and an overall depth across the vertical profile not exceeding 80 % of the design joint thickness in order that the tie may be accommodated within the mortar joint	Should transmit the forces (tensile and compressive) arising from applied loads on the masonry and allow only small deformations along the axis of the tie. Should permit the following differential movements arising from moisture and thermal expansion or contraction of the two leaves during the service life of the wall without generating undue stress in the wall: a) differential movements in the vertical direction between the two leaves of up to 12 mm; b) differential movements in the horizontal direction between the two leaves of up to 6 mm. Should be provided with a profile designed to cause water to drip off before reaching the inner leaf. This should function even when the tie is at a slope of 5° downwards from outer leaf to inner leaf.	As for inner leaf connection
Type 2, 3 and 4	As type 1	As for type 1, but should be sufficiently ductile or deformable to permit adjustments by hand of up to 25 mm in the vertical direction during construction, to allow for differences in the height of the bedding courses of the two leaves.	As type 1

C.4 Dimensions and tolerances of ties

The design of the tie should be such as to meet the requirements of this British Standard, whilst allowing for the tolerances normal in buildings, such as the width of the cavity varying from the stated design cavity width.

The embedment length, or lengths, should be long enough to allow a depth of embedment of at least 62.5 mm for a tie placed centrally in a wall of the maximum thickness designated for the particular tie type and built in accordance with BS 5628-3, so that the actual embedment after allowing for building tolerances will be at least 50 mm.

C.5 Performance of wall ties

The tensile and compressive load capacity of tie types should be equal to, or greater than, the specified load capacity for a specified embedment length, but should not be less than the figures given in Table C.3, when the embedment length of each end of the tie is taken to be 50 mm.

NOTE When the declared values of tie capacity are based on an embedment length greater than 50 mm, tie capacity values applicable to an embedment of 50 mm should be assessed for the purpose of satisfying the minima given in Table C.3.

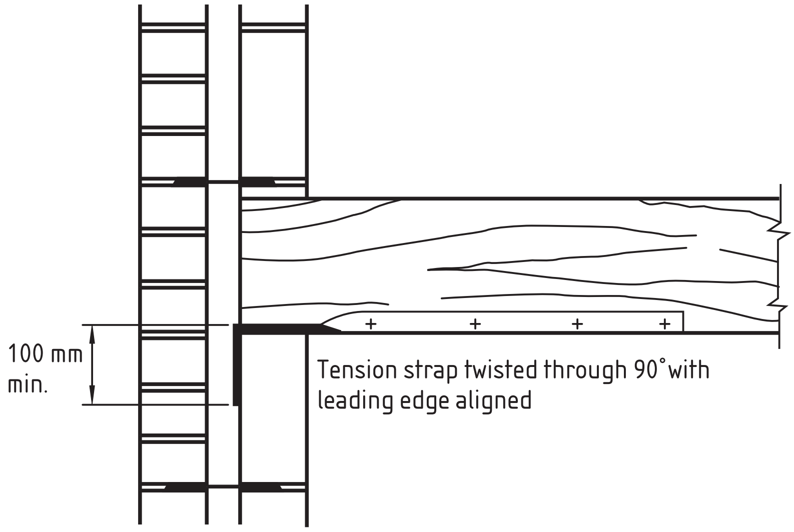
Table C.3 — Minimum declared tensile load capacity and compression load capacity for tie type

Tie type	**Minimum mortar class and designation**	**Tensile load capacity** N	**Compressive load capacity** N
1	M12 (i)	5 000	5 000
1	M2 (iv)	2 500	2 500
2	M2 (iv)	1 800	1 300
3	M2 (iv)	1 100	800
4	M2 (iv)	650	450

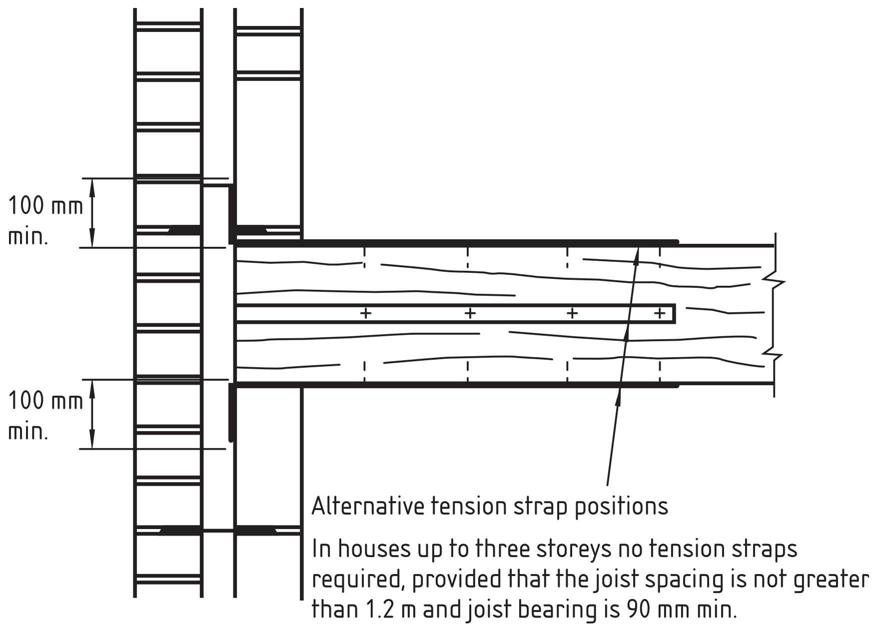
Annex D (informative) Connections to floors and roofs by means of tension straps and joist hangers

Figures D.1 to D.14 illustrate connections that may be used to provide horizontal lateral restraint in accordance with **24.2.3**. Tension straps or joist hangers and their fixings should conform to BS EN 845-1 and be capable of resisting lateral loads as specified in **24.2.2**. The connections should be provided at intervals of not more than 2 m in houses of not more than three storeys, and not more than 1.25 m for all storeys in all other buildings. In the illustrations, floors are generally shown; however, the same details are applicable to roofs.

Joist hangers and fixings as detailed in Figure D.4 and Figure D.5, or tension straps having a nominal cross section of 30 mm × 5 mm with fixings capable of resisting lateral loads, as specified in **24.2.2**, may be assumed to have adequate strength in buildings of up to six storeys in height.



a)



b)

Figure D.1 — Timber floor bearing directly on to wall

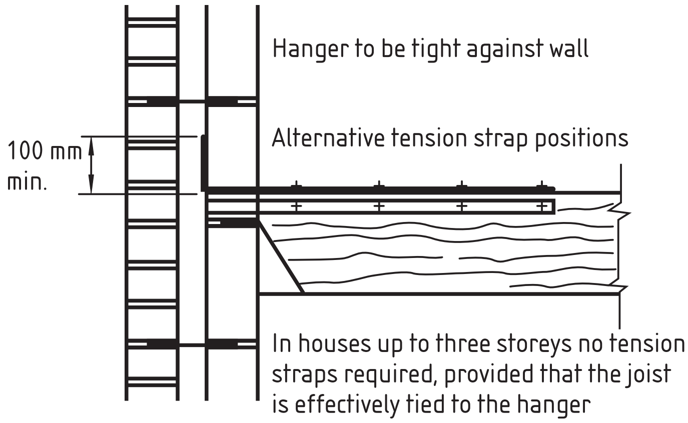


Figure D.2 — Timber floor using typical joist hanger

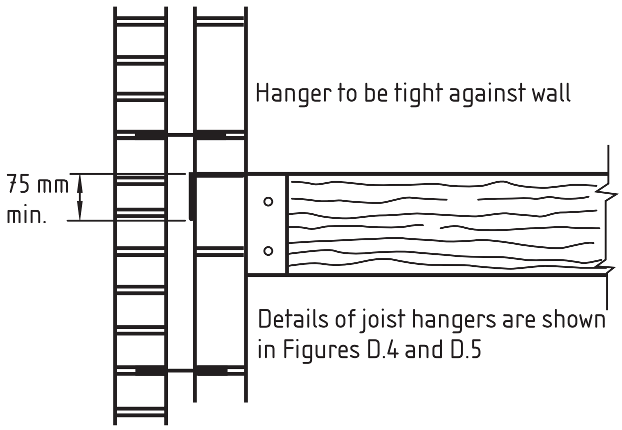
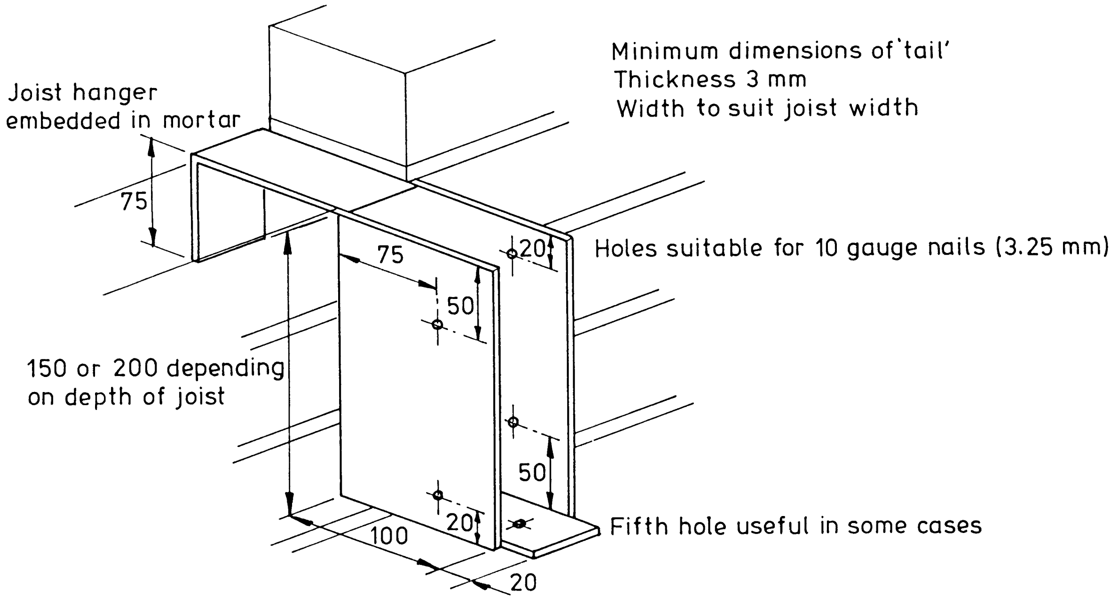
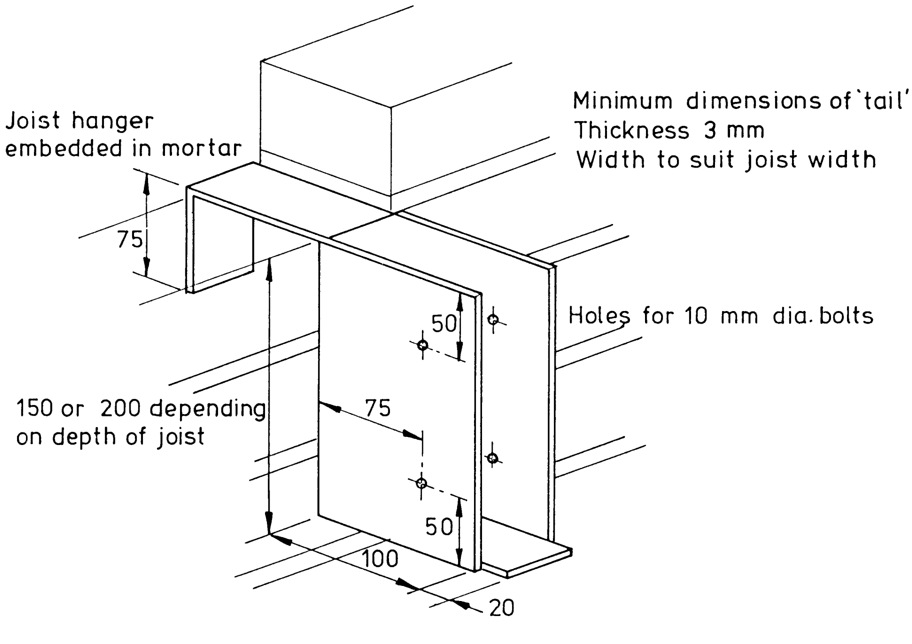


Figure D.3 — Timber floor using nailed or bolted joist hangers acting as tie



All dimensions are in millimetres.

Figure D.4 — Joist hanger as tie: nailed form



All dimensions are in millimetres.

Figure D.5 — Joist hanger as tie: bolted form

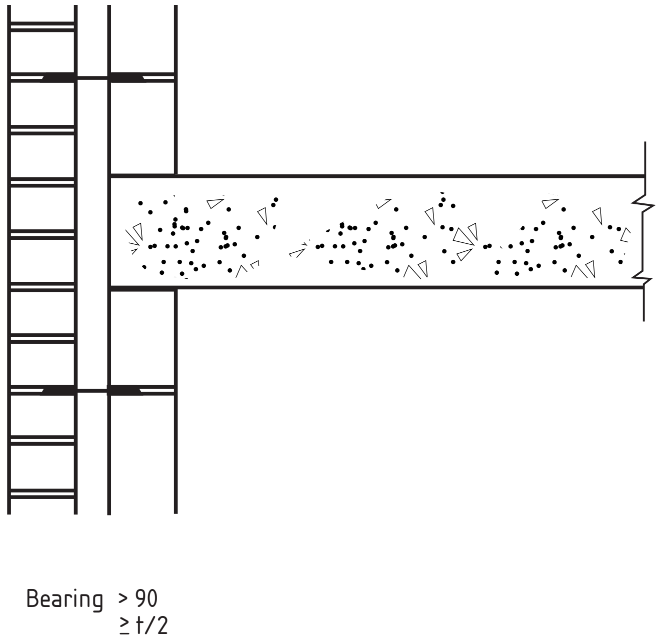


Figure D.6 — In-situ concrete floor bearing directly on to wall

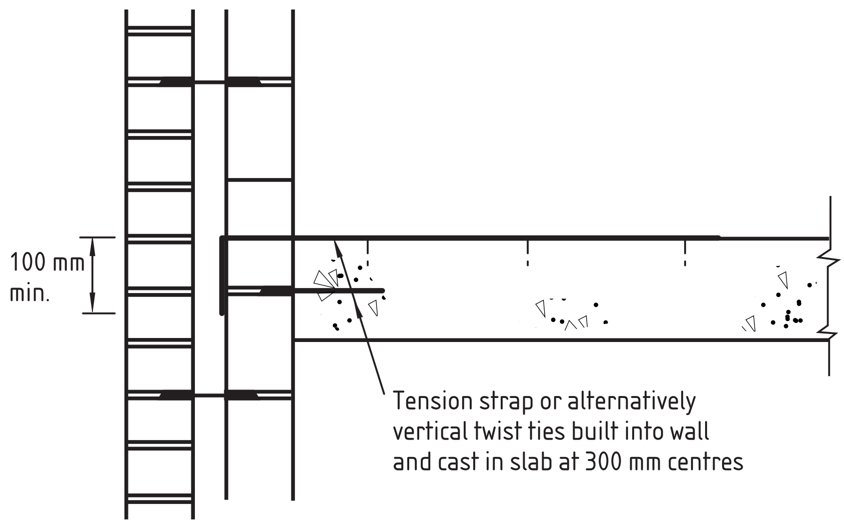


Figure D.7 — In-situ concrete floor abutting external cavity wall

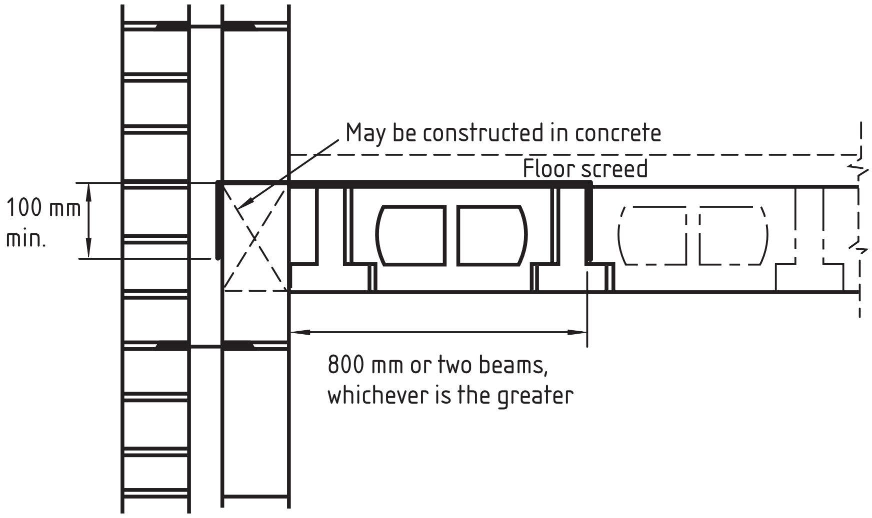


Figure D.8 — Beam and pot floor abutting external cavity wall

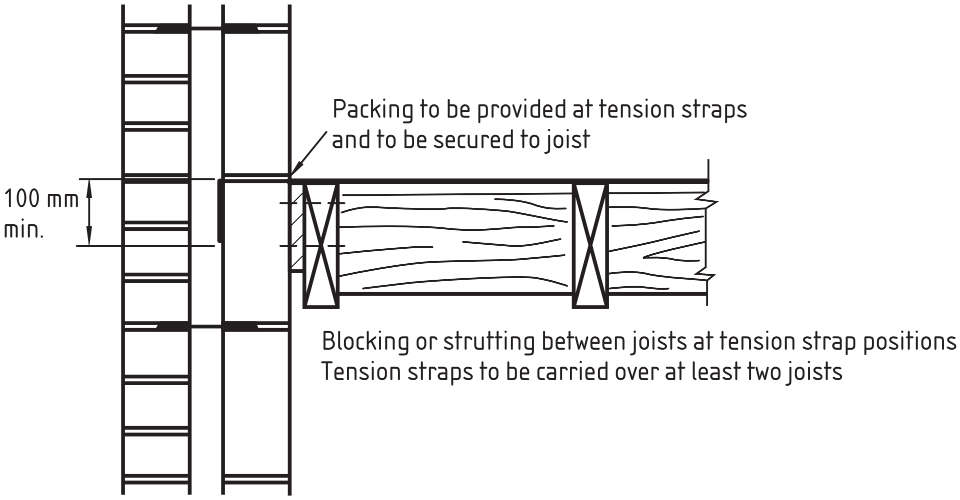


Figure D.9 — Timber floor abutting external cavity wall

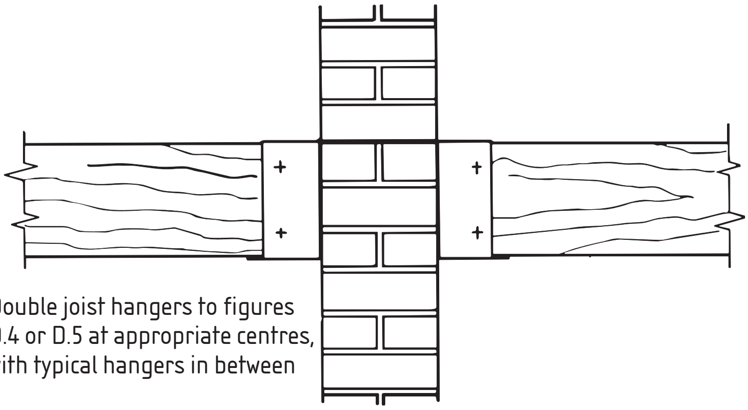


Figure D.10 — Timber floor using double joist hanger acting as tie

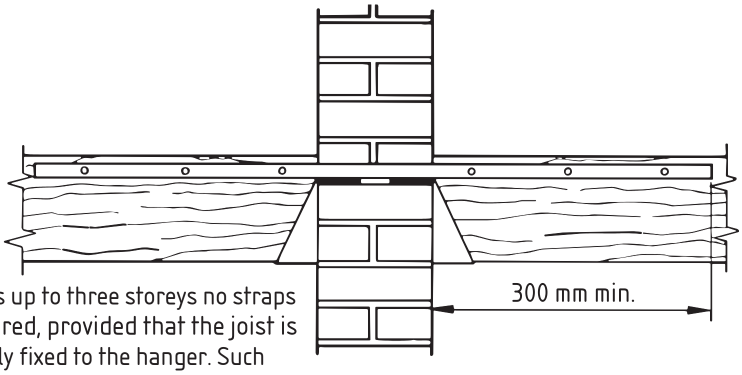


Figure D.11 — Timber floor using typical joist hanger

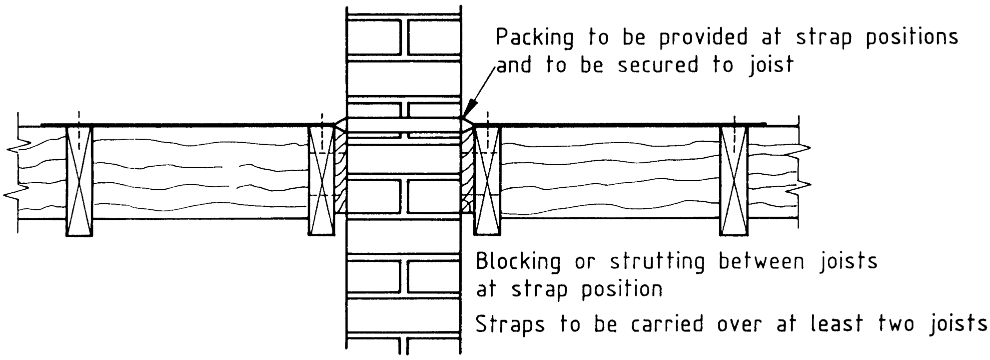


Figure D.12 — Timber floor abutting internal wall

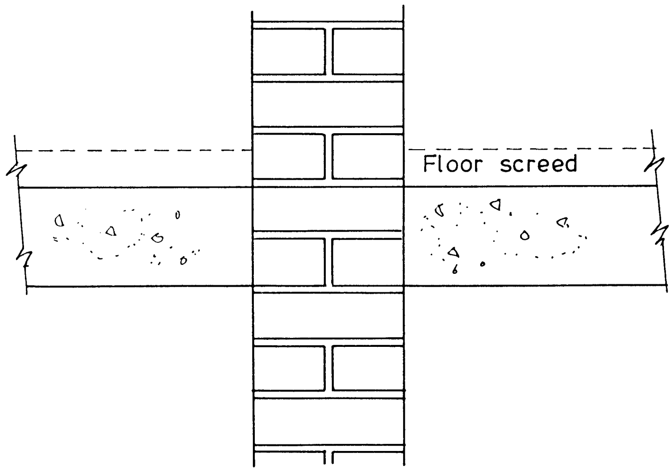


Figure D.13 — In-situ floor abutting internal wall

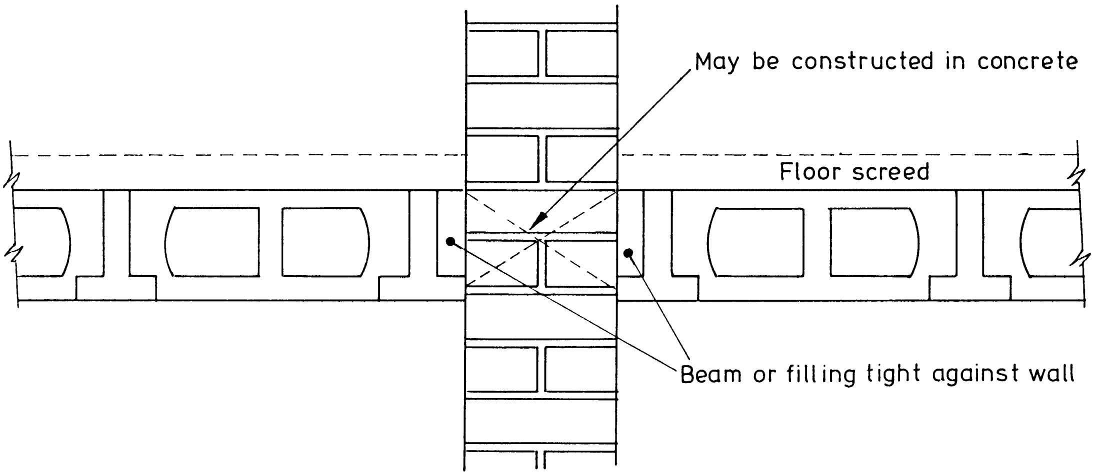


Figure D.14 — Beam and pot floor abutting internal wall

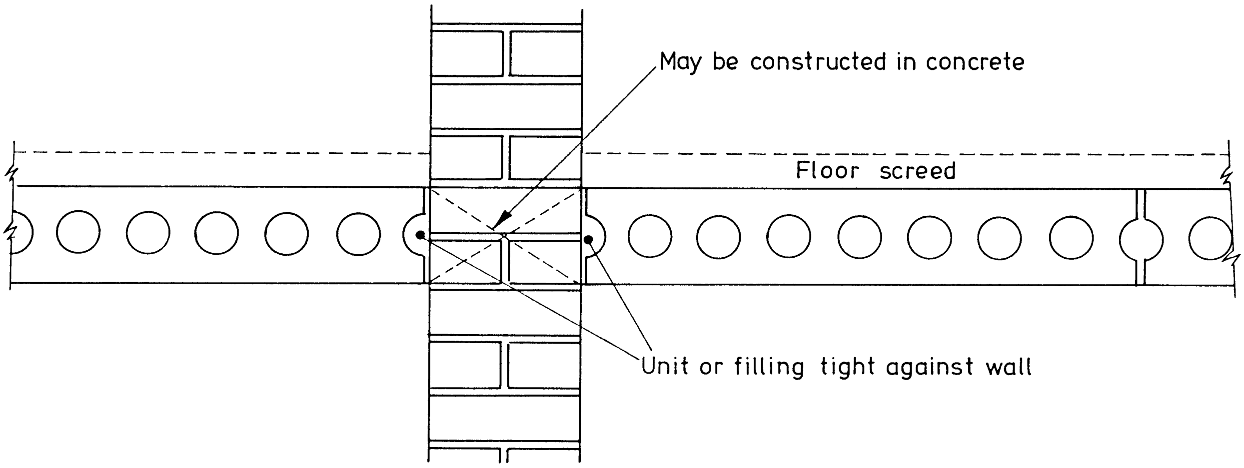


Figure D.15 — Precast units abutting internal wall

Annex E (informative)
Laterally loaded panels of irregular shape, or those containing openings

When irregular shaped panels, or those with substantial openings are to be designed, it will often be possible to divide them into sub-panels that can then be calculated in accordance with Clause **32** (see Figure E.1). Alternatively, an analysis using a recognised method of obtaining bending moments in flat plate, e.g. yield line or finite element may be used and these can then be used instead of the moments obtained from the coefficients given in Table 8.

Small openings in panels will have little effect on the strength of the panel in which they occur, and they can be ignored. When suitable timber or metal frames are built into openings, the strength of the frame, taken in conjunction with the masonry panel, will often be sufficient to replace the strength lost by the area of the opening. Such cases will have to be decided by the designer, as guidance is beyond the scope of this British Standard.

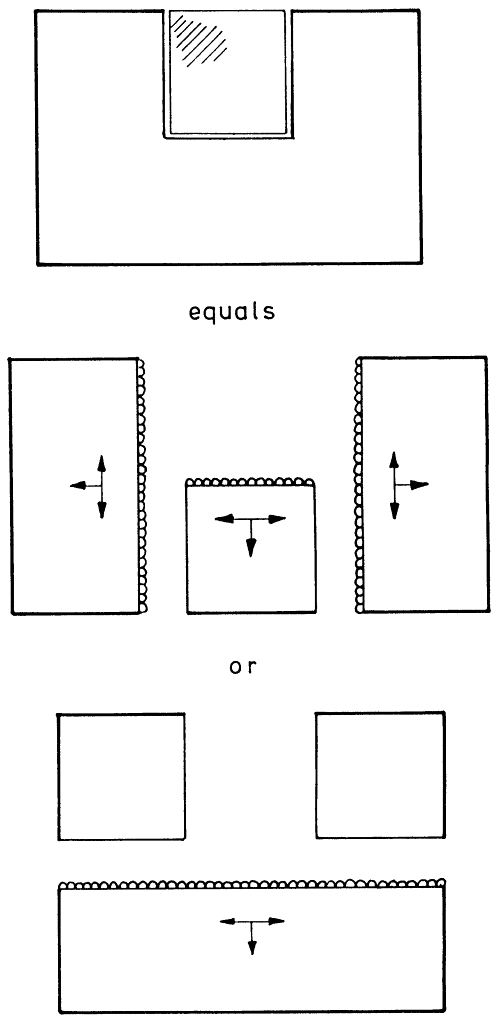


Figure E.1 — Dividing a panel into parts to allow for openings

Bibliography

[1] BS 5502, *Buildings and structures for agriculture — Part 22: Code of practice for design, construction and loading*.

[2] BS 8110, *Structural use of concrete*.

[3] *Civil Engineering Code of Practice No. 2 (Earth retaining structures)*: 1951
available for consultation at the library of the Institution of Civil Engineers.

Index

In this index references are to clause, appendix, table and figure numbers.

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