British Standard

BRITISH STANDARD

BS 5628-1: 1992

Incorporating Amendments Nos. 1 and 2

Code of practice for use of masonry —

Part 1: Structural use of unreinforced masonry

British Standards

Committees responsible for this British Standard

The preparation of this British Standard was entrusted by the Technical Committee B/525, Building and Civil Engineering Structures, to Subcommittee B/525/6, Use of masonry, upon which the following bodies were represented:

Association of Consulting Engineers
Autoclaved Aerated Concrete Products Association
Brick Development Association
British Ceramic Research Ltd.
British Masonry Society
British Precast Concrete Federation Ltd.
Calcium Silicate Brick Association
Concrete Block Association
Department of the Environment (Building Research Establishment)
Department of the Environment (Construction Directorate)
Department of the Environment (Property Services Agency)
Institution of Civil Engineers
Institution of Structural Engineers
National House Building Council
Royal Institute of British Architects

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Contents

Foreword

This Part of BS 5628 has been prepared under the direction of Technical Committee B/525, Building and Civil Engineering Structures. It supersedes BS 5628-1:1978, which is withdrawn.

This edition of BS 5628-1 introduces technical changes, but it does not reflect a full review or revision of the standard which will be undertaken in due course.

This Part uses limit state philosophy, in accordance with practice in other codes. The 1948, 1964 and 1970 editions of CP 111 were in permissible stress terms, and a section on reinforced masonry walls was included. Reinforced masonry has been excluded from this Part of BS 5628, as the subject is dealt with in Part 2. Plain concrete walls are covered in BS 8110.

The use of limit state enables the degree of risk to be varied by the choice of different partial safety factors. It has been recognized in this Part that masonry is built to differing standards of workmanship and, therefore, the partial safety factors have reflected this.

In view of the widespread demand for guidance on the design of walls to resist lateral loading, a section on this subject was incorporated in the 1978 edition. This was based on a large volume of research work. This new edition includes a comprehensive method of design for proposed cantilever walls for certain types of single storey buildings under wind loading.

It has become necessary, in structural design, to ensure explicitly that a building will not collapse catastrophically as the result of misuse or accident. The recommendations dealing with this requirement reflect the available theoretical and research studies on the behaviour of elements and structures under extreme conditions. Whilst some aspects of the work require further study, the guidance given represents a balanced combination of facts and engineering judgement.

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

This document comprises a front cover, an inside front cover, pages i to iv, pages 1 to 71 and a back cover.

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Section 1. General

1 Scope

This Part of BS 5628 gives recommendations for the structural design of unreinforced masonry constructed of brick masonry, block masonry, masonry of square dressed natural stone, and random rubble masonry.

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, 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 References

The titles of the standards publications referred to in this code are listed on page 70.

3 Definitions

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

3.1
actual dimension
either the work size of the unit or, where applicable for solid walls, the sum of the work size of the units together with the work size of the joints between them

3.2
category 1 building
a building having four storeys (including basement storeys), or less

3.3
category 2 building
a building having five storeys (including basement storeys), or more

3.4
characteristic load
ideally, where the load acts unfavourably, the load which has a probability of not more than 5 % of being exceeded or, where the load acts favourably, the load which has a probability of at least 95 % of being exceeded. In practice, the load obtained from the appropriate British Standard

3.5
characteristic strength of masonry
the value of the strength of masonry below which the probability of test results falling is not more than 5 %

3.6
compressive strength of structural units
for the normal category of manufacturing control, the strength of a sample tested according to the appropriate British Standard

NOTE When manufacturing control is within the special category as defined in **27.2.1.2**, compressive strength is taken as the acceptance limit, therein defined.

3.7
column
an isolated vertical loadbearing member whose width is not more than four times its thickness

3.8
design load
the characteristic load multiplied by a partial safety factor for loads

3.9
design strength
the characteristic strength divided by a partial safety factor for material strength

3.10
effective height or length
the height or length of a wall, pier or column assumed for calculating the slenderness ratio

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

3.12
laterally loaded wall panels
walls subjected mainly to loads normal to the face of the wall

3.13
lateral support
the support, in relation to a wall or pier, which will restrict movement in the direction of the thickness of the wall or, in relation to a column, which will restrict movement in the direction of its thickness or width. Lateral supports may be horizontal or vertical

3.14
loadbearing walls
walls primarily designed to carry an imposed vertical load in addition to their own weight

3.15
masonry
an assemblage of structural units, either laid in-situ or constructed in prefabricated panels, in which the structural units are bonded and solidly put together with mortar or grout. Masonry may be reinforced or unreinforced

3.16
orthogonal ratio
the 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

3.17
pier
a member which forms an integral part of a wall, in the form of a thickened section placed at intervals along the wall

3.18
protected member
a structural member capable of resisting a specified pressure

NOTE See Section 5.

3.19
slenderness ratio
the ratio of the effective height or length to the effective thickness

3.20
structural units
bricks or blocks or square dressed natural stone

3.21 Types of wall

3.21.1
single-leaf wall
a wall of bricks or blocks laid to overlap in one or more directions and set solidly in mortar

3.21.2
double-leaf (collar-jointed) wall
two parallel single-leaf walls, with a space between not exceeding 25 mm, filled solidly with mortar and so tied together as to result in common action under load

3.21.3
cavity wall
two parallel single-leaf walls, usually at least 50 mm apart, and effectively tied together with wall ties, the space between being left as a continuous cavity or filled with non-loadbearing material

3.21.4
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

3.21.5
faced wall
a wall in which the facing and backing are so bonded as to result in common action under load

3.21.6
veneered wall
a wall having a facing which is attached to the backing, but not so bonded as to result in common action under load

3.22
wallette
a small masonry panel constructed for test purposes

4 Symbols

The following symbols are used in this code.

A	horizontal cross-sectional area
B	width of a bearing under a concentrated load
b	width of column
E_u	worst credible earth or water lateral load (see clause **21**)
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_y	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 a span in accidental damage 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
ψ_m reduction factor for strength of mortar
ψ_u unit reduction factor

5 Alternative materials and methods of design and construction

Where materials and methods are used that are not referred to by this code, their use is not discouraged, provided that the materials comply with the requirements of the appropriate British Standards 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 code.

Where materials or methods are used that are not referred to by this code 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 and characteristic flexural strength of masonry are set out in **A.2** and **A.3** respectively.

Section 2. Materials, components and workmanship

6 General

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

7 Structural units

The following structural units should comply with the appropriate clauses in the relevant British Standard:

Calcium silicate (sandlime and flintlime) bricks	BS 187
Clay bricks	BS 3921
Dimensions of bricks of special shapes and sizes	BS 4729
Stone masonry	BS 5628-3
Precast concrete masonry units	BS 6073-1
Reconstructed stone masonry units	BS 6457
Clay and calcium silicate modular bricks	BS 6649

Materials that have been previously used should not be re-used in masonry construction unless they have been thoroughly cleaned and conform to the code recommendations for similar new materials.

8 Laying of structural units

8.1 General

Structural units will normally be laid on their bed faces. In circumstances 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 the appropriate clauses of BS 187, BS 6073-1 and BS 3921.

8.2 Bricks with frogs

Bricks should normally be laid on a full bed mortar[1)] with the frog or larger frog uppermost, which should be filled with mortar as the work proceeds.

8.3 Perforated bricks

Perforated bricks and bricks defined as solid according to BS 187 and BS 3921, but having perforations passing through them, should be laid on a full width bed of mortar[1)]. The bedding mortar for divided joint units should be laid by means of a mortar tray in accordance with the brickmaker's recommendations. The perforations should only be filled with mortar as required in BS 5628-3 or when specified by the designer.

8.4 Hollow and cellular blocks

8.4.1 ***Hollow clay blocks***

Hollow clay blocks may have horizontal or vertical perforations. The blocks should be laid on a full bed of mortar[1)], except those with vertical perforations which are designed to be laid with a divided bed joint in accordance with the manufacturer's recommendations.

8.4.2 ***Hollow and cellular concrete blocks***

Hollow and cellular concrete blocks complying with the requirements of BS 6073-1 are normally laid on a full bed of mortar[1)]. Occasionally such blocks are laid with the bed joint separated into two parallel strips.

9 Rate of laying

The maximum height that should normally be built in a day is 1.5 m.

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.

[1)] When bed joints are raked out for pointing allowance should be made in design for the resulting loss of strength.

11 Pallet slips

Timber pallet slips or other material for fixings should not be built into walls without the approval of the designer.

12 Damp proof courses

Damp proof courses should comply with the requirements of one of the British Standards, as appropriate, specified in clause **4.7** of BS 5628-3:2001. In addition the recommendations of **5.6** of BS 8215 should be taken into account.

Designers should pay particular attention to the characteristics of the materials chosen for damp proof courses. Materials which 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 described in DD 86-1 and DD 86-2.

13 Wall ties

Wall ties should comply with BS 1243 or meet the recommendations of DD 140-2 (see also **36.2**).

In situations of severe exposure, or where required by building regulations, suitable stainless steel or non-ferrous ties should be used.

14 Cements

Cement should comply with the requirements of one of the following British Standards: BS 12, BS 146, BS 4027 and BS 5224.

The use of high alumina cement is not permitted.

15 Mortars

15.1 General

The mixing and use of mortars should be in accordance with the recommendations given in BS 5628-3. The proportion of the materials and mean compressive strengths required are given in Table 1. When testing is required, it should be in accordance with **A.1**.

15.2 Ready-mixed mortars

Ready-mixed lime : sand for mortar should comply with the requirements of BS 4721. The appropriate addition of cement should be gauged on site.

Wet ready-mixed retarded cement : lime : sand mixes should be used only with the written permission of the designer.

16 Colouring agents

Pigments should comply with the requirements of BS 1014 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.

17 Plasticizers

Plasticizers should comply with the requirements of BS 4887 and should be used only with the written permission of the designer.

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

18 Frost inhibitors

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

Table 1 — Requirements for mortar

	Mortar designation	Type of mortar (proportion by volume)			Mean compressive strength at 28 days	
		Cement : lime : sand	Masonry cement : sand	Cement : sand with plasticizer	Preliminary (laboratory) tests	Site tests
					N/mm^2	N/mm^2
↑ Increasing strength; ↓ Increasing ability to accommodate movement, e.g. due to settlement, temperature and moisture changes	(i)	1 : 0 to ¼ : 3	—	—	16.0	11.0
	(ii)	1 : ½ : 4 to 4½	1 : 2½ to 3½	1 : 3 to 4	6.5	4.5
	(iii)	1 : 1 : 5 to 6	1 : 4 to 5	1 : 5 to 6	3.6	2.5
	(iv)	1 : 2 : 8 to 9	1 : 5½ to6½	1 : 7 to 8	1.5	1.0
Direction of change in properties is shown by the arrows		Increasing resistance to frost attack during construction → Improvement in bond and consequent resistance to rain penetration ←				

blank

Section 3. Design: objectives and general recommendations

19 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 assessment of the characteristic compressive strength may be based either on the results of laboratory tests carried out on representative wall panels, as outlined in **A.2**, or on the information given in clause **23**.

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 based on those adopted in BS 8110.

γ_f is 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.

20 Stability

20.1 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 4 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. $0.015G_k$) above any level [see clause **22**b) and clause **22**c)];

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

20.2 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 code. However, in order to establish section sizes and strengths which 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 clause **21** and clause **22**, 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.

20.3 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.

20.4 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.

21 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 and BS 6399-3.

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 6399-2.

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 **22**).

22 Design loads: partial safety factor, γ_f

When using the design relationship for the ultimate limit state given in sections 4 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*

design dead load = $0.9G_k$ or $1.4G_k$

design imposed load = $1.6Q_k$

design worst credible earth and water lateral load = $1.2E_u$

b) *Dead and wind load*

design dead load = $0.9G_k$ or $1.4G_k$

design wind load = $1.4W_k$

design worst credible earth and water lateral load = $1.2E_u$

In the particular case of freestanding walls and laterally loaded wall panels, whose removal 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*

design dead load	=	$1.2G_k$
design imposed load	=	$1.2Q_k$
design wind load	=	$1.2W_k$
design worst credible earth and water lateral load	=	$1.2E_u$

d) *Accidental damage* (see **37**)

design dead load	=	$0.95G_k$ or $1.05G_k$
design imposed load	=	$0.35Q_k$ except that, in the case of buildings used predominantly for storage, or where the imposed load is of a permanent nature, $1.05Q_k$ should be used
design wind load	=	$0.35W_k$

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 water lateral load (see clause **21**); 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 Civil Engineering Code of Practice No.2 1951, the appropriate safety factor γ_f for design worst credible earth and water lateral 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.

23 Characteristic compressive strength of masonry, f_k

23.1 Normal masonry

The characteristic compressive strength, f_k, of any masonry may be determined by tests on wall specimens, following the procedures laid down in **A.2**.

For normally bonded masonry, defined in terms of the shape and compressive strength of the structural units and the designation of the mortar (see Table 1), the values given in Table 2 inclusive 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, facilitated by the graphs [Figure 1a), Figure 1b), Figure 1c) and Figure 1d)].

Table 2a) applies to masonry built with standard format bricks complying with the requirements of BS 187, BS 6073-1 or BS 3921.

Table 2b) applies to masonry built with structural units with a ratio of height to least horizontal dimension of 0.6.

Table 2c) applies to structural units, other than solid concrete blocks, with a ratio of height to least horizontal dimension of between 2.0 and 4.0, and makes due allowance for the enhancement in strength resulting from the unit shape.

Table 2d) applies to solid concrete blocks, i.e. those without cavities, with a ratio of height to least horizontal dimension of between 2.0 and 4.0, and makes due allowance for the enhancement in strength resulting from the unit shape.

23.1.1 ***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:

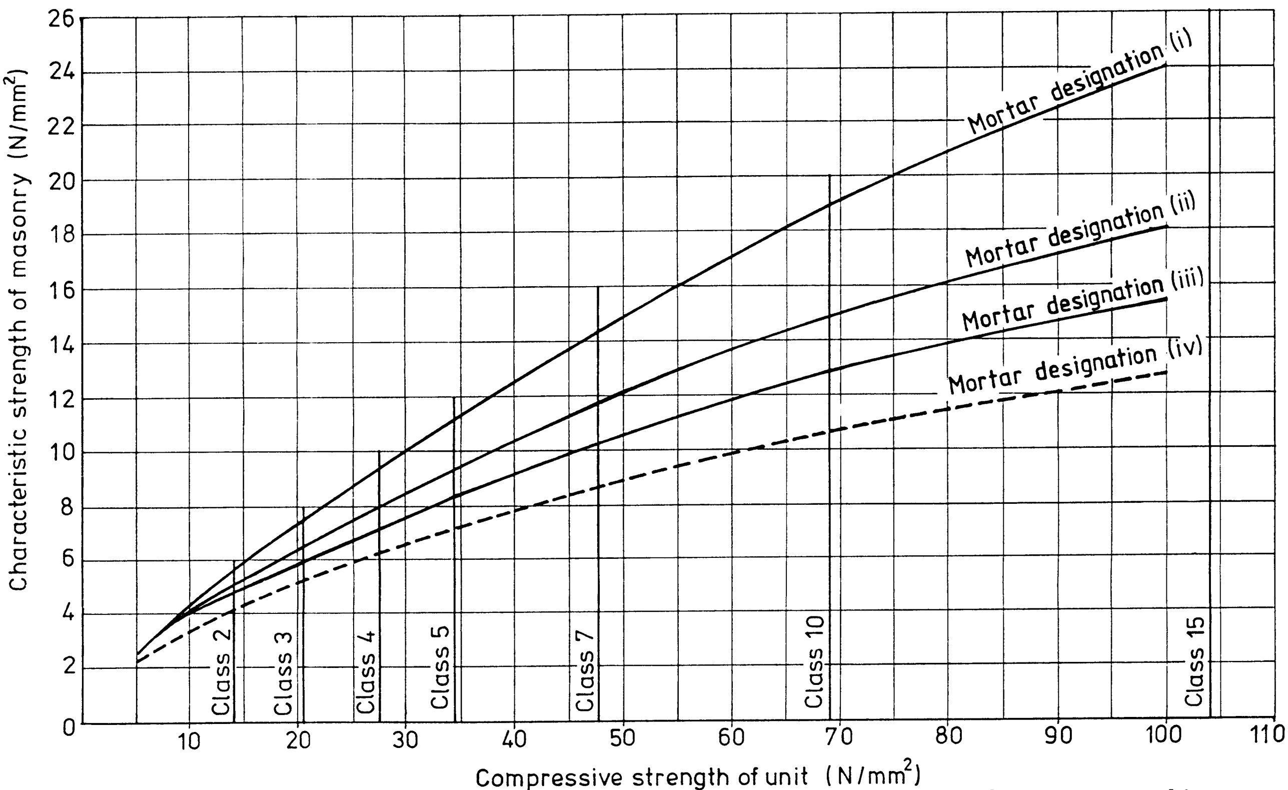
$$(0.70 + 1.5A)$$

where

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

23.1.2 ***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.



Interpolation for classes of loadbearing bricks not shown on the graph may be used for average crushing strengths intermediate between those given on the graph, as described in clause **10** of BS 3921 : 1974 and clause **7** of BS 187 : 1978.

Figure 1a) — Characteristic compressive strength of brick masonry, f_k [see Table 2a)]

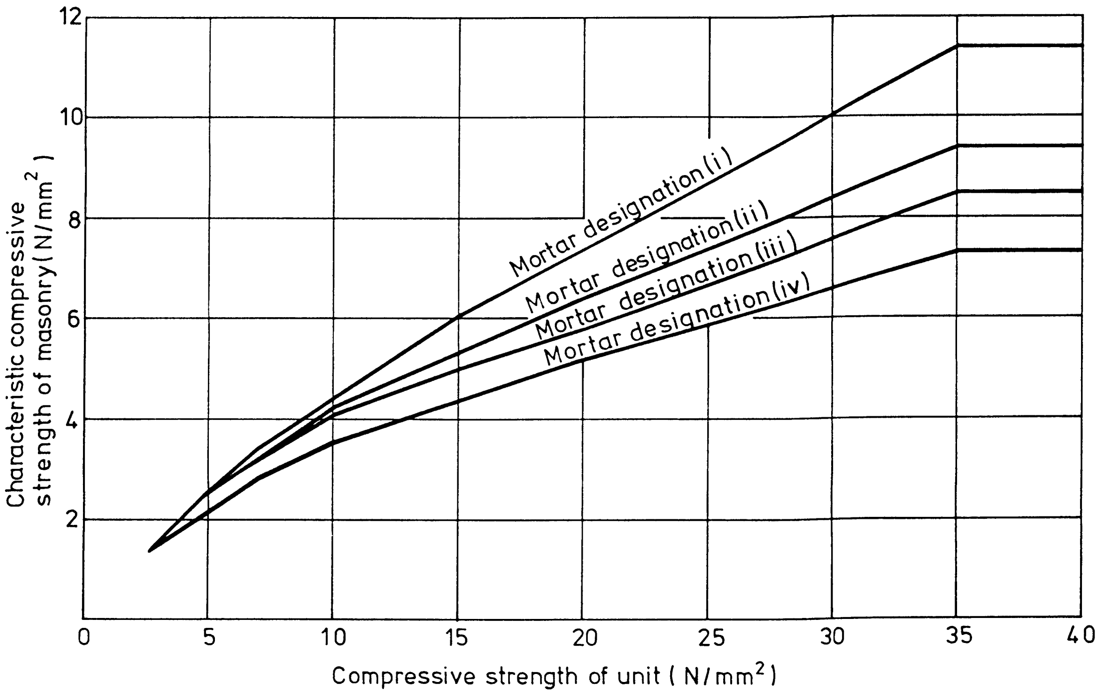


Figure 1b) — Characteristic compressive strength, f_k, of block masonry constructed with blocks having a ratio of height to least horizontal dimension of 0.6 [see Table 2b)]

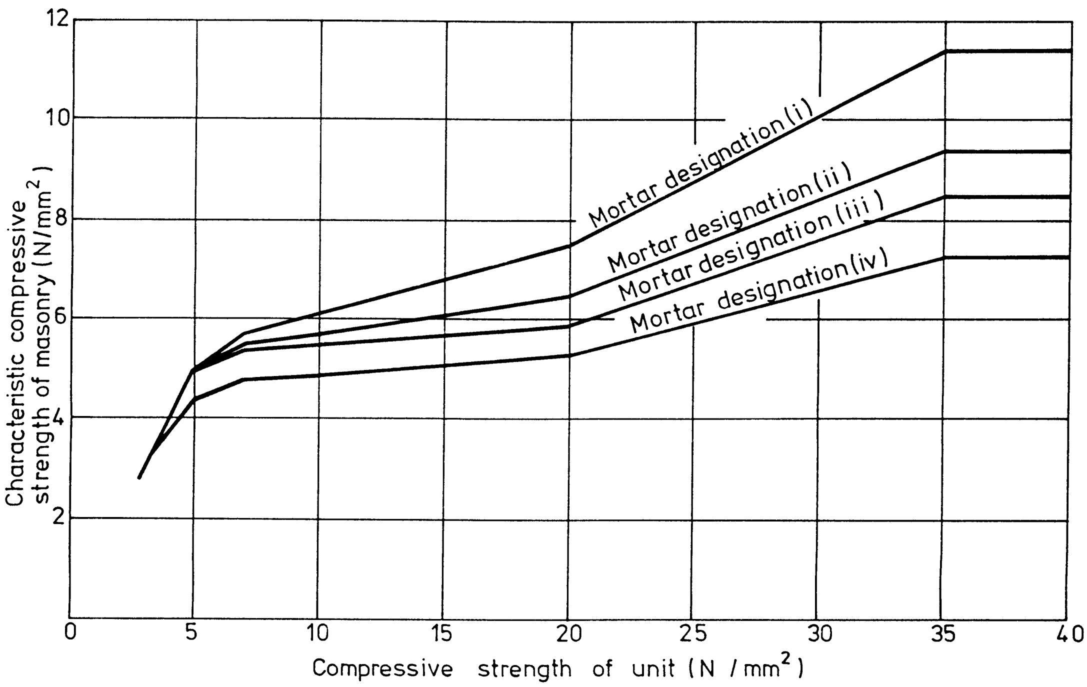


Figure 1c) — Characteristic compressive strength, f_k, of block masonry constructed with hollow blocks having a ratio of height to least horizontal dimension of between 2.0 and 4.0 [see Table 2c)]

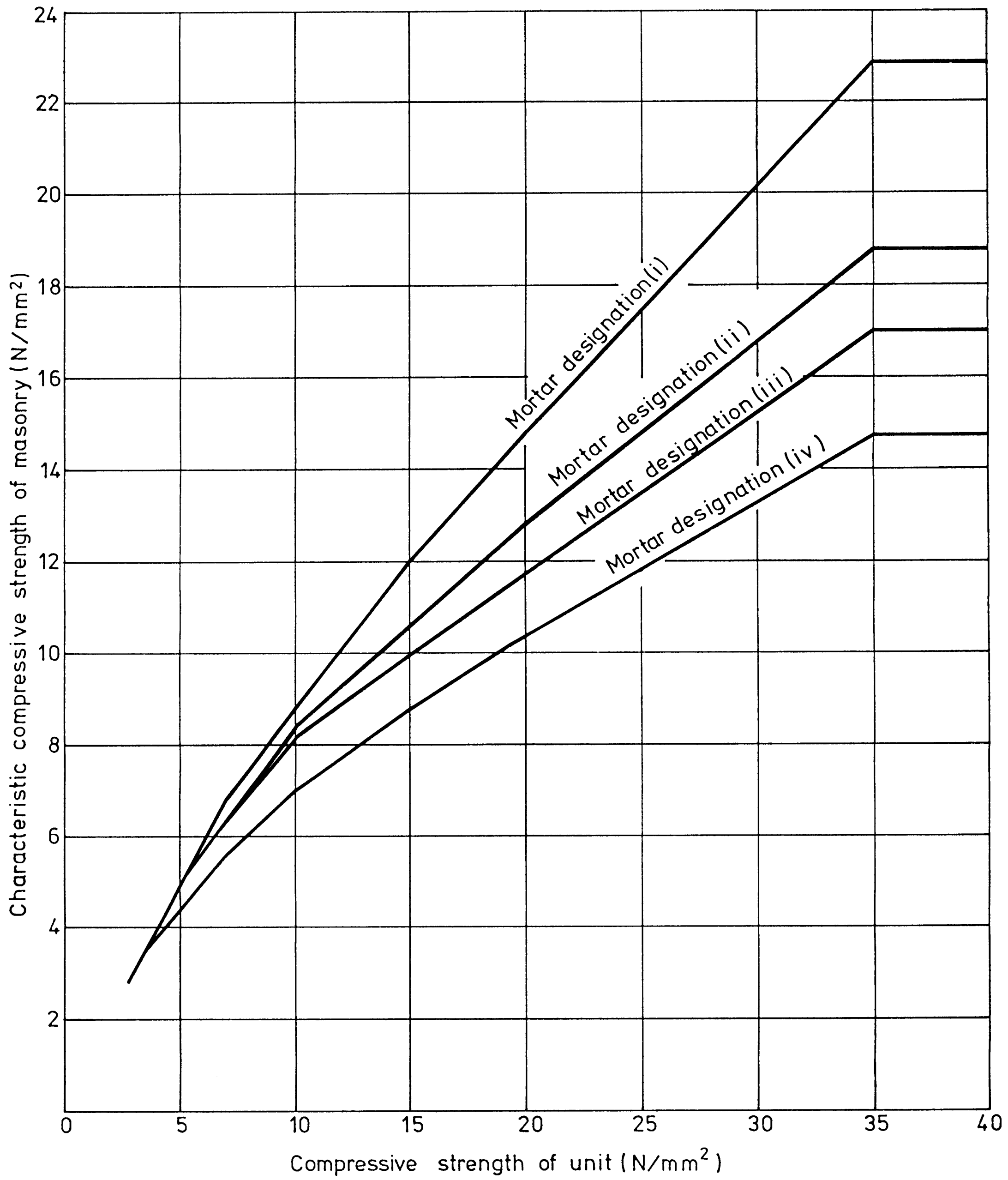


Figure 1d) — Characteristic compressive strength, f_k, of block masonry constructed from solid concrete blocks having a ratio of height to least horizontal dimension of between 2.0 and 4.0 [see Table 2d)]

Table 2 — Characteristic compressive strength of masonry, f_k, in N/mm²

a) Constructed with standard format bricks

Mortar designation	Compressive strength of unit (N/mm²)								
	5	**10**	**15**	**20**	**27.5**	**35**	**50**	**70**	**100**
(i)	2.5	4.4	6.0	7.4	9.2	11.4	15.0	19.2	24.0
(ii)	2.5	4.2	5.3	6.4	7.9	9.4	12.2	15.1	18.2
(iii)	2.5	4.1	5.0	5.8	7.1	8.5	10.6	13.1	15.5
(iv)	2.2	3.5	4.4	5.2	6.2	7.3	9.0	10.8	12.7

b) Constructed with blocks having a ratio of height to least horizontal dimension of 0.6

Mortar designation	Compressive strength of unit (N/mm²)							
	2.8	**3.5**	**5.0**	**7.0**	**10**	**15**	**20**	**35 or greater**
(i)	1.4	1.7	2.5	3.4	4.4	6.0	7.4	11.4
(ii)	1.4	1.7	2.5	3.2	4.2	5.3	6.4	9.4
(iii)	1.4	1.7	2.5	3.2	4.1	5.0	5.8	8.5
(iv)	1.4	1.7	2.2	2.8	3.5	4.4	5.2	7.3

c) Constructed with hollow blocks having a ratio of height to least horizontal dimension of between 2.0 and 4.0

Mortar designation	Compressive strength of unit (N/mm²)							
	2.8	**3.5**	**5.0**	**7.0**	**10**	**15**	**20**	**35 or greater**
(i)	2.8	3.5	5.0	5.7	6.1	6.8	7.5	11.4
(ii)	2.8	3.5	5.0	5.5	5.7	6.1	6.5	9.4
(iii)	2.8	3.5	5.0	5.4	5.5	5.7	5.9	8.5
(iv)	2.8	3.5	4.4	4.8	4.9	5.1	5.3	7.3

d) Constructed from solid concrete blocks having a ratio of height to least horizontal dimension of between 2.0 and 4.0

Mortar designation	Compressive strength of unit (N/mm²)							
	2.8	**3.5**	**5.0**	**7.0**	**10**	**15**	**20**	**35 or greater**
(i)	2.8	3.5	5.0	6.8	8.8	12.0	14.8	22.8
(ii)	2.8	3.5	5.0	6.4	8.4	10.6	12.8	18.8
(iii)	2.8	3.5	5.0	6.4	8.2	10.0	11.6	17.0
(iv)	2.8	3.5	4.4	5.6	7.0	8.8	10.4	14.6

23.1.3 ***Walls constructed in modular bricks***

For 90 mm wide × 90 mm high modular bricks complying with the requirements of BS 6073-1 or as detailed in DD 34 or DD 59, the values of f_k in Table 2a) may be multiplied by the following factors:

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

b) other thicknesses: 1.10.

23.1.4 ***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 have been obtained from wall tests carried out in accordance with **A.2**.

23.1.5 ***Hollow block walls***

When walls are built of hollow blocks 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 2b) and Table 2c).

23.1.6 ***Solid concrete block walls***

When walls are built of solid concrete blocks, i.e. those without any cavities, 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 2b) and Table 2d).

23.1.7 ***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 were solid (see **23.1.6**), 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.

23.1.8 ***Natural stone masonry***

Natural stone masonry should be designed on the basis of solid concrete blocks of an equivalent compressive strength. Where masonry is constructed from large, carefully shaped pieces with relatively thin joints, its loadbearing capacity is more closely related to the intrinsic strength of the stone than is the case where small structural units are used. Design stresses in excess of those obtained from this code may be allowed in such massive stone masonry, provided that the designer is satisfied that the properties of the stone warrant an increase.

23.1.9 ***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 designation (iv).

23.2 Structural units laid other than on the normal bed face

Where structural units 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.

23.3 Perforated bricks and hollow blocks

The compressive strength of 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), Table 2b) and Table 2c). 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 Table 2b) and Table 2c), but the design strength of the wall should be reduced by the ratio of the bedded area to the net area of block.

24 Characteristic flexural strength of masonry, f_{kx}

24.1 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 half the values in Table 3 may be allowed in direct tension 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 which has been proved by tests (see DD 86-1) to permit the joint to transmit tension or if it is of bricks complying with the requirements of BS 743.

24.2 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 **A.3**.

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 2.8 N/mm^2 and 7.0 N/mm^2 in a wall of given thickness.

25 Characteristic shear strength of masonry, f_v

The characteristic shear strength of masonry, f_v, in the horizontal direction of the horizontal plane (see Figure 2) may be taken as $0.35 + 0.6g_A$ N/mm² with a maximum of 1.75 N/mm² for walls built in mortar designations (i) and (ii) or $0.15 + 0.6g_A$ N/mm² with a maximum of 1.4 N/mm² for walls built in mortar designations (iii) and (iv):

where

g_A is the design vertical load per unit area of wall cross section due to the vertical loads calculated from the appropriate loading condition specified in clause **22**.

The characteristic shear strength f_v of bonded masonry in the vertical direction of the vertical plane (see Figure 2) may be taken as:

a) for brick:

0.7 N/mm² [for mortar designations (i) and (ii)];

0.5 N/mm² [for mortar designations (iii) and (iv)];

b) for dense aggregate solid concrete block with a minimum strength of 7 N/mm²:

0.35 N/mm² [for mortar designations (i), (ii) and (iii)].

26 Coefficient of friction

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

27 Partial safety factors for material strength, γ_m

27.1 General

The value of γ_m to be used in the application of the design procedures given in Section 4 and Section 5 should be related to the quality control exercised.

27.2 Quality control

The value of γ_m adopted should be commensurate with the degree of control exercised both during the manufacture of the structural 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 **27.2.1** and **27.2.2**.

27.2.1 ***Manufacturing control***

27.2.1.1 *Normal category*

Normal category should be assumed when the supplier is able to meet the requirements for compressive strength in the appropriate British Standard, but does not meet the requirements for the special category detailed in **27.2.1.2**.

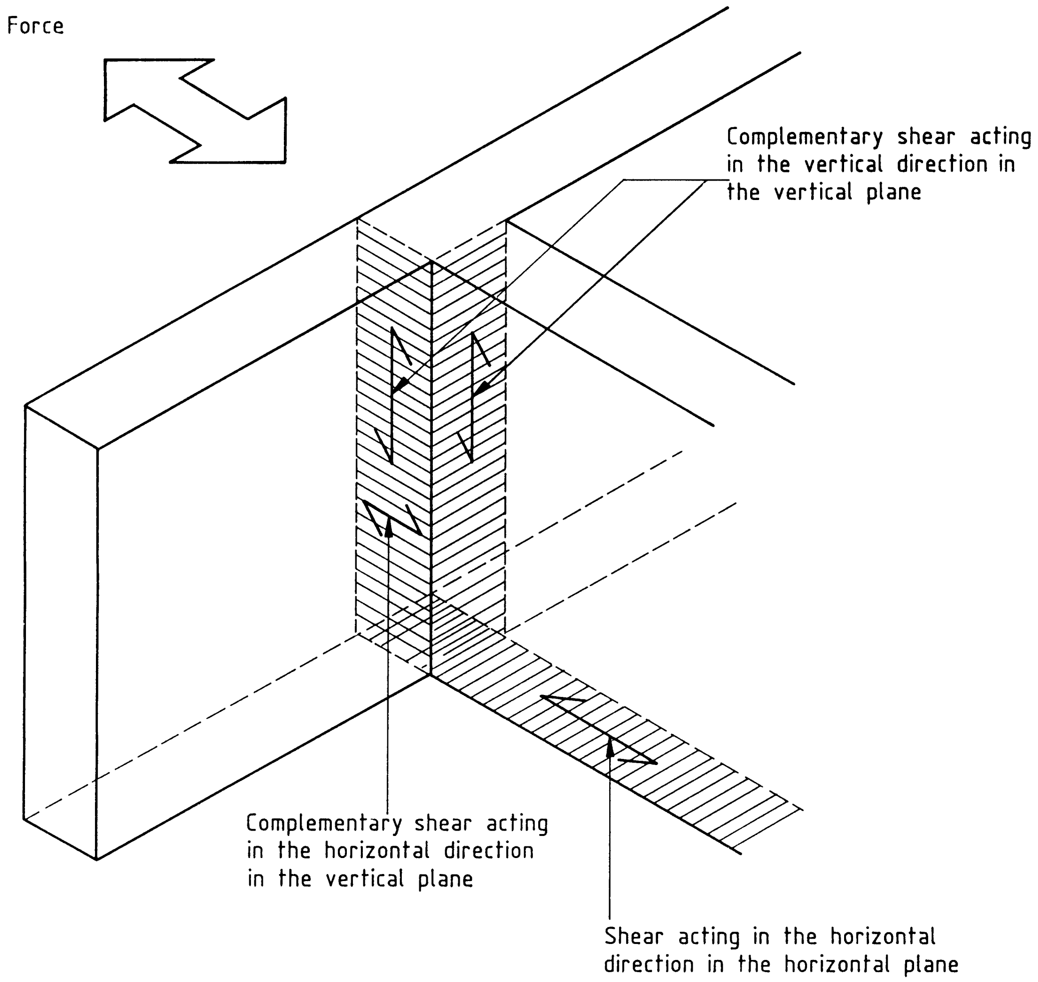
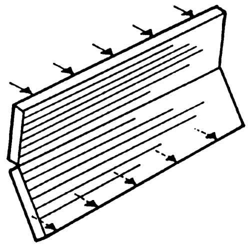
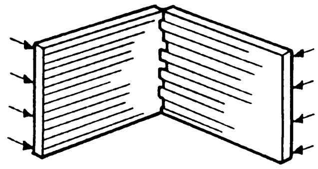


Figure 2 — Shear forces acting in the horizontal and vertical planes

Table 3 — Characteristic flexural strength of masonry, f_{kx}, in N/mm²

	Plane of failure parallel to bed joints 			Plane of failure perpendicular to bed joints 		
Mortar designation	**(i)**	**(ii) and (iii)**	**(iv)**	**(i)**	**(ii) and (iii)**	**(iv)**
Clay bricks having a water absorption						
less than 7 %	0.7	0.5	0.4	2.0	1.5	1.2
between 7 % and 12 %	0.5	0.4	0.35	1.5	1.1	1.0
over 12 %	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 (solid or hollow) of compressive strength in N/mm²:						
2.8, 3.5, 7.0 used in walls of thickness[a] up to 100 mm	0.25		0.2	0.40, 0.45, 0.60		0.4, 0.4, 0.5
2.8, 3.5, 7.0 used in walls of thickness[a] 250 mm	0.15		0.1	0.25, 0.25, 0.35		0.2, 0.2, 0.3
10.5, 14.0 and over used in walls of any thickness[a]	0.25		0.2	0.75, 0.90[b]		0.6, 0.7

[a] 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.
[b] When used with flexural strength in parallel direction, assume the orthogonal ratio $\mu = 0.3$.

27.2.1.2 *Special category*

Special category may be assumed where the manufacturer:

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

b) operates a quality control scheme, the results of which can be made available to demonstrate to the satisfaction of the purchaser that the acceptance limit is consistently being met in practice, with the probability of failing to meet the limit being never greater than that stated in **27.2.1.2**a).

27.2.2 *Construction control*

27.2.2.1 *Normal category*

Normal category should be assumed whenever the work is carried out following the recommendations for workmanship in Annex A of BS 5628-3:2001, or BS 8000-3 including appropriate supervision and inspection.

27.2.2.2 *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 4a);

b) preliminary compressive strength tests carried out on the mortar to be used, in accordance with **A.1**, indicate compliance with the strength requirements given in Table 1 and regular testing of the mortar used on site, in accordance with **A.1**, shows that compliance with the strength requirements given in Table 1 is being maintained.

27.3 Values of γ_m for normal and accidental loads

The value of γ_m to be used in clause **32** and clause **34** should be obtained from Table 4a). The value of γ_m to be used in clause **36**, except where otherwise stated, should be obtained from Table 4b). When considering the probable effects of misuse or accident (Section 5), the values given in Table 4a) and Table 4b) may be halved, except where otherwise required in **37.1.1**.

Table 4a) — Partial safety factors for material strength — Compression, γ_m

		Category of construction control	
		Special	**Normal**
Category of manufacturing control of structural units	**Special**	2.5	3.1
	Normal	2.8	3.5

Table 4b) — Partial safety factors for material strength — Flexure, γ_m

Category of construction control	
Special	**Normal**
2.5	3.0

Where wall tests in accordance with **A.2** and **A.3** have been carried out, the γ_m factors to be applied to the characteristic strengths determined in accordance with **A.2** and **A.3** may be taken as 0.9 times the values given in Table 4a) and Table 4b), provided the masonry conforms in all respects to the recommendations given in this code.

27.4 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 mortar 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.

27.5 Values of γ_m for use with ties

The partial safety factory to be applied to the strength of wall ties (**36.2**) should be 3.0. When considering the probable effects of misuse or accidental damage, this value may be halved.

Section 4. Design: detailed considerations

28 Consideration of slenderness of walls and columns

28.1 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.

28.2 Lateral support

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.

28.2.1 ***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.

28.2.2 ***Horizontal lateral supports***

28.2.2.1 Simple resistance to lateral movement may be assumed in the case of houses of not more than three storeys where timber floor members, spaced apart at a distance of not more than 1.2 m, are connected by suitable joist hangers effectively fixed to the joist, as indicated in Appendix C. 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 Appendix C.

28.2.2.2 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.

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

28.2.3 ***Vertical lateral supports***

28.2.3.1 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 **28.2.1** and evenly distributed throughout the height at not more than 300 mm centres.

28.2.3.2 Enhanced resistance to lateral movement may be assumed where an intersecting or return wall as described in **28.2.3.1** is properly bonded to the supported wall or loadbearing leaf of a cavity wall.

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

28.3 Effective height or length

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 **28.3.1** or **28.3.2**.

28.3.1 ***Effective height***

28.3.1.1 *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) the clear distance between lateral supports which provide simple resistance to lateral movement.

28.3.1.2 *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.

28.3.1.3 *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.

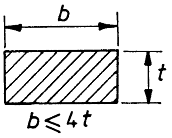
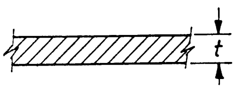
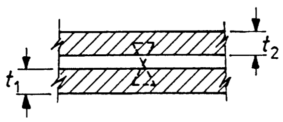
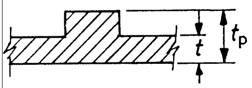
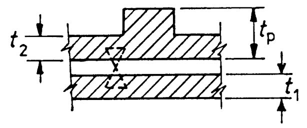
Column	Single-leaf wall	Cavity wall	Walls stiffened by piers	
			Single-leaf	Cavity
Plan shapes				
				
Effective thickness				
t or b, depending on direction of bending	t	the greatest of: a) 2/3 ($t_1 + t_2$); or b) t_1; or c) t_2	$t \times k$	the greatest of: a) 2/3 ($t_1 + Kt_2$); or b) t_1; or c) Kt_2
			where K is the stiffness coefficient from Table 5	

Figure 3 — Effective thickness of columns and walls

28.3.1.4 *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.

28.3.2 ***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.

28.4 Effective thickness

The effective thickness of a wall, column or pier is given in **28.4.1** and **28.4.2** and is illustrated in Figure 3.

28.4.1 ***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.

28.4.2 ***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.

29 Special types of wall

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.

29.1 Cavity walls

29.1.1 *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 **23.1.2** and **28.4**).

29.1.2 *Minimum thickness of leaves*

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

29.1.3 *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 **5.5.4.2.5** of BS 5628-3:2001).

29.1.4 *Selection and strength of ties*

The leaves of a cavity wall should be tied together with wall ties suitable for the type of construction.

The design compressive and tensile resistance of the ties, relevant to the nominal cavity width, should exceed the design loads to which they will be subjected.

Table 6 gives guidance on the selection of ties for normal applications.

29.1.5 *Density and positioning of ties*

The density (number of 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. 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 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.

29.1.6 *Embedment of ties*

The minimum embedment of a tie in the mortar joint should be 50 mm in each leaf. The length of the ties should be sufficient to give the minimum embedment having regard to normal site tolerances for cavity width and centering of the tie; suitable minimum lengths are given in Table 6.

Table 6 — Selection of wall ties: types and lengths

		Permissible type of tie		
Least leaf thickness (one or both) mm	**Nominal cavity width** mm	**Tie[a] length** mm	**Shape name in accordance with BS 1243**	**Type number in accordance with DD 140-2[b]** mm
75	75 or less	200	Butterfly, double triangle or vertical twist	Types 1, 2, 3 or 4, selected on the basis of the design loading and design cavity width. Prescriptive rules for selection, and a model calculation, are given in DD 140-2.
90	76 to 90	225	Double triangle[c] vertical twist or	
90	91 to 100	225	Double triangle[d] vertical twist or	
90	101 to 125	250	Vertical twist	
90	126 to 150	275	Vertical twist	
90	151 to 175	300	Vertical twist	
90	176 to 300	[a]	Vertical twist style[e]	

[a] This column gives the tie lengths, in 25 mm increments, that best meet the performance requirement that the embedment depth will be not 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. For cavities wider than 180 mm, calculate the length as the structural cavity width plus 125 mm and select the nearest stock length. The designer may vary these in particular circumstances, provided that the design requirements continue to be met.

[b] The strength and stiffness of masonry/masonry ties in accordance with DD 140 ranges from type 1, the stiffest, to type 4 the least stiff. For ties to BS 1243 the vertical twist is the stiffest and the butterfly the least stiff.

[c] The minimum length requirement exceeds the maximum length specified under BS 1243 but 225 mm double triangle format ties, which otherwise comply with the requirements of BS 1243, should be suitable.

[d] Double triangle ties of shape similar to those in BS 1243, having a strength to satisfy type 2 of DD 140-2, are manufactured. Specialist tie manufacturers should be consulted if 225 mm long double triangle format ties are needed for 91 mm to 100 mm cavities.

[e] BS 1243 only covers tie lengths up to 305 mm but longer ties, which otherwise comply with the standard, have been found to give adequate performance in up to 300 mm wide cavities.

29.2 External cavity walls

29.2.1 *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.

29.2.2 *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.

29.2.3 *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 appropriate ties which are able to accommodate the expected movement, the fixing of windows to outer leaves, the provision of soft joints under cills and the use of suitable wall head details.

29.2.4 *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).

29.3 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 **29.2.3**. 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 **29.4**).

29.4 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.

29.5 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.

29.6 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 obtained from clause **23** is multiplied by 0.9.

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) Flat metal wall ties of cross-sectional area 20 mm × 3 mm at centres not exceeding 450 mm both vertically and horizontally, or an equivalent mesh at the same vertical centres, are provided.

6) The minimum embedment of the ties into each leaf is 50 mm.

7) The vertical collar joint between the two leaves is solidly filled with mortar as the work proceeds.

29.7 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 complies with the requirements detailed in **29.6**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.

30 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.

31 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.

32 Walls and columns subjected to vertical loading

32.1 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 4 and the wall strength should be calculated in accordance with **32.2**.

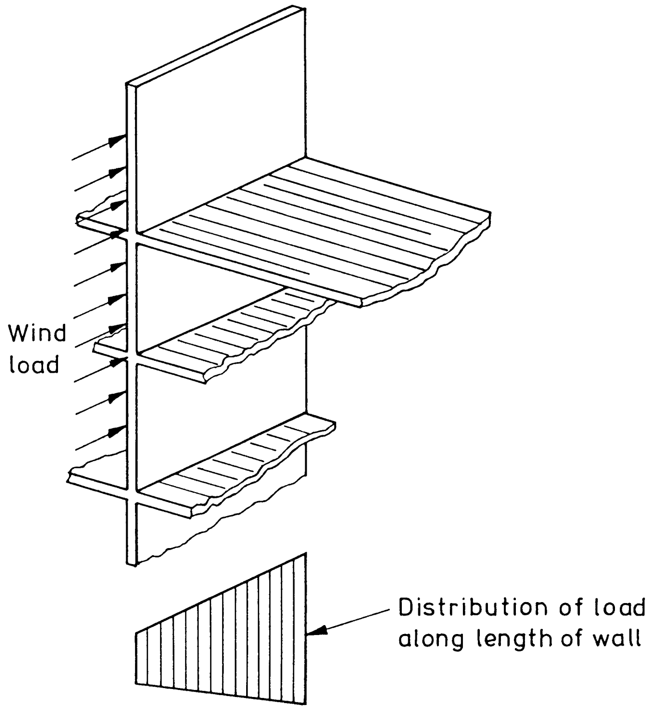


Figure 4 — Load distribution from loading eccentric to plane of wall

32.2 Design strength of masonry

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.

32.2.1 *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 **23**;
- γ_m is the appropriate partial safety factor for the material obtained from clause **27** Table 4a);
- t is the thickness of the wall.

Table 7 — Capacity reduction factor, β

Slenderness ratio h_{ef}/t_{ef}	Eccentricity at top of wall, e_x			
	Up to 0.05t (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 Appendix B.

32.2.2 ***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 **32.2.1**. 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 Appendix 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 Appendix B, deriving additional eccentricities about both axes.

32.2.3 ***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 **32.2.1** or **32.2.2** as appropriate, in which case t_{ef} in Table 7 is the effective thickness of the cavity wall or column.

33 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:

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

where

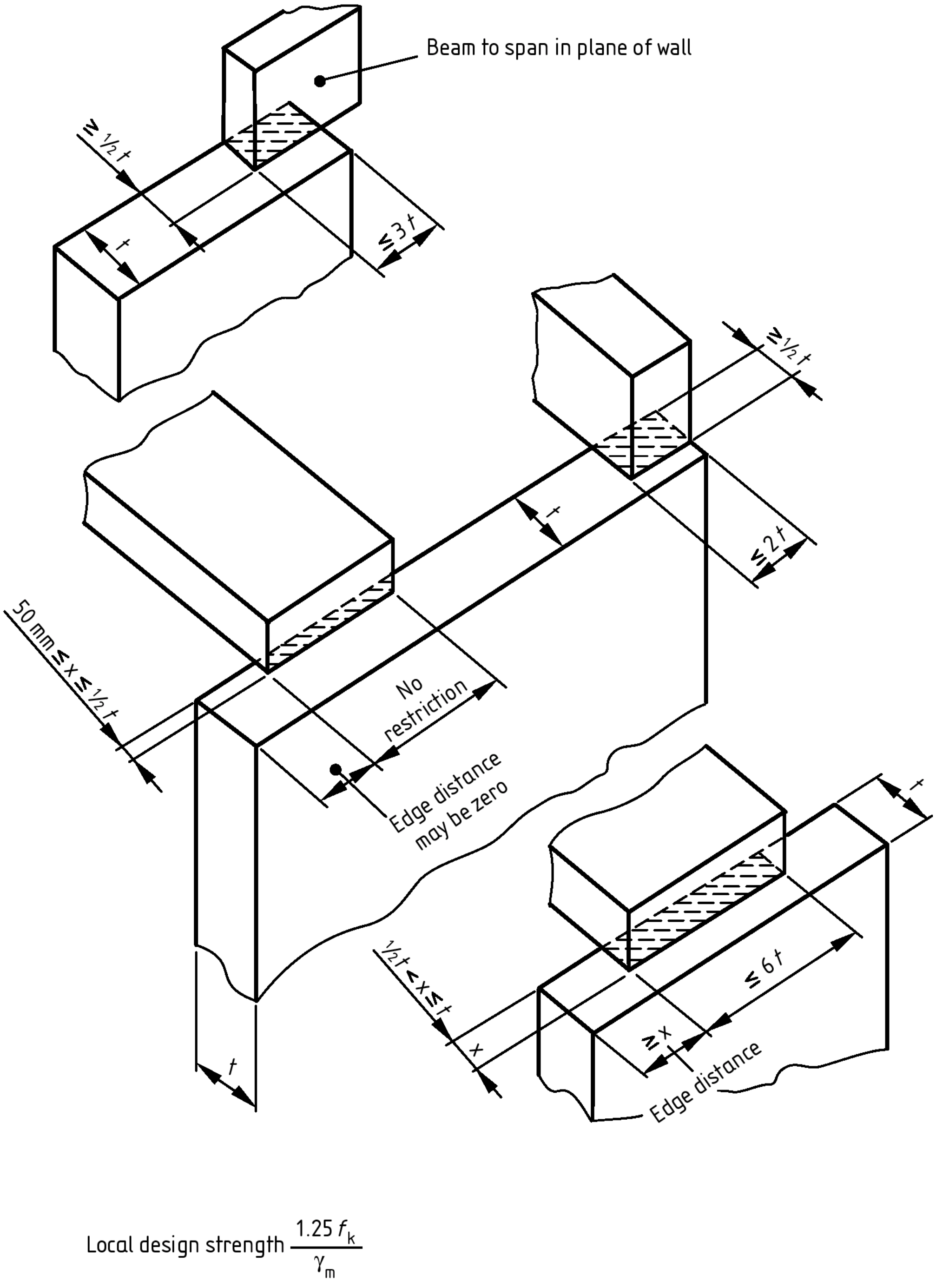
v_h is the shear stress produced by the horizontal design load calculated as acting uniformly over the horizontal cross-sectional area of the wall;

γ_{mv} is the partial safety factor for material strength in shear (**27.4**);

f_v is the characteristic shear strength of the masonry (clause **25**).

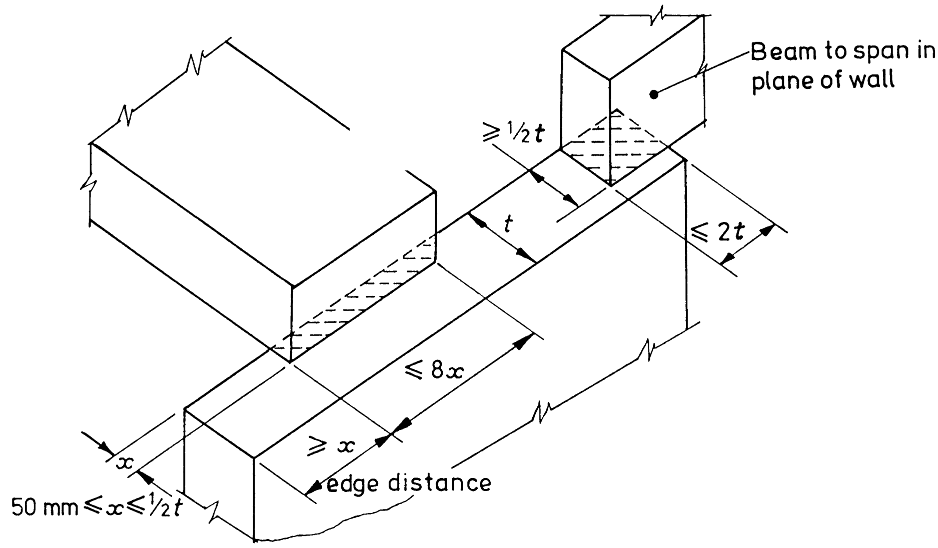
34 Concentrated loads: stresses under and close to a bearing

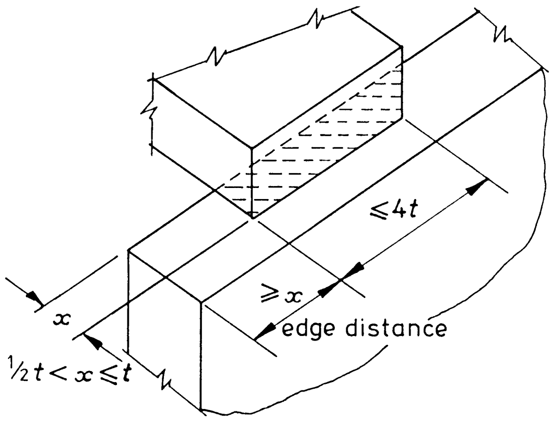
Increased local stresses may be permitted beneath the bearing of a concentrated load of a purely local nature, such as beams, columns, lintels, etc. provided either that the element applying the load is sensibly rigid, or that a suitable spreader is introduced. The concentrated load may be assumed to 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 5(c)], and dispersed in two planes within a zone contained by lines extending downwards at 45° from the edges of the loaded area.



(a) Bearing type 1

Figure 5 — Concentrated loads: types of bearing

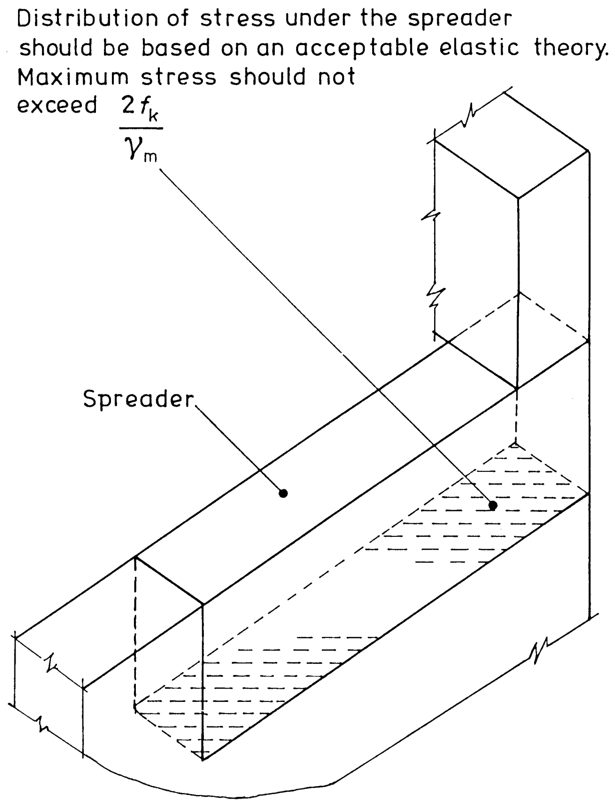




Local design strength $\frac{1.5 f_k}{\gamma_m}$

(b) Bearing type 2

Figure 5 — Concentrated loads: types of bearing (*continued*)



(c) Bearing type 3

Figure 5 — Concentrated loads: types of bearing (*concluded*)

The effect of the local load combined with stresses due to other loads [see Figure 6(a)] 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 5(a)] or $1.5f_k/\gamma_m$ in the case of bearing type 2 [Figure 5(b)];

b) at a distance of $0.4h$ below the bearing where the design strength should be calculated in accordance with clause **32**;

where

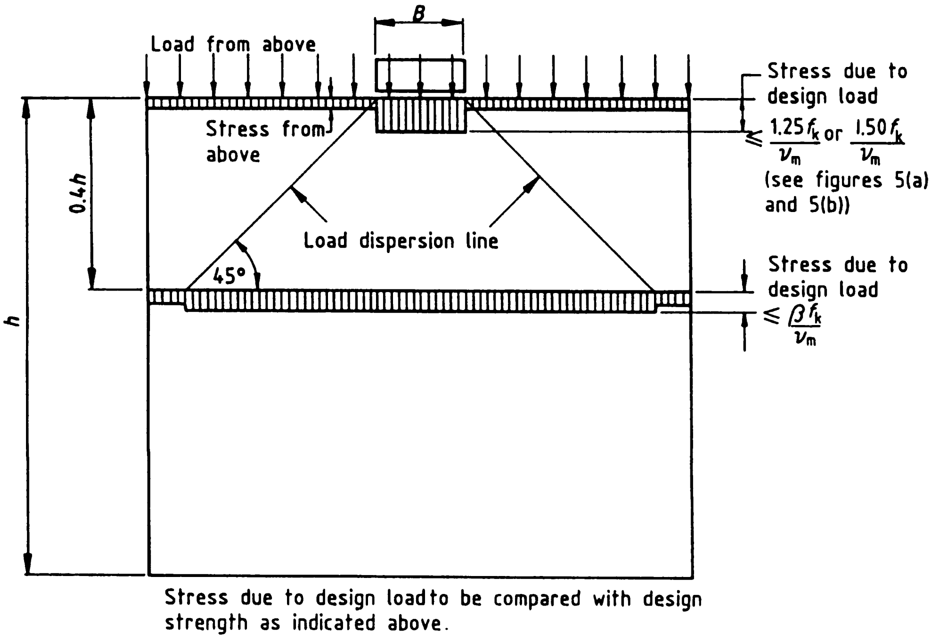
f_k	is the characteristic strength of the masonry;
γ_m	is the appropriate partial safety factor for the material [clause **27**, Table 4a)];
h	is the clear height of the wall;
β	is the capacity reduction factor from Table 7.

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 it its plane [see Figure 5(c)], the maximum stress at the bearing combined with stresses due to other loads should not exceed $2.0f_k/\gamma_m$.

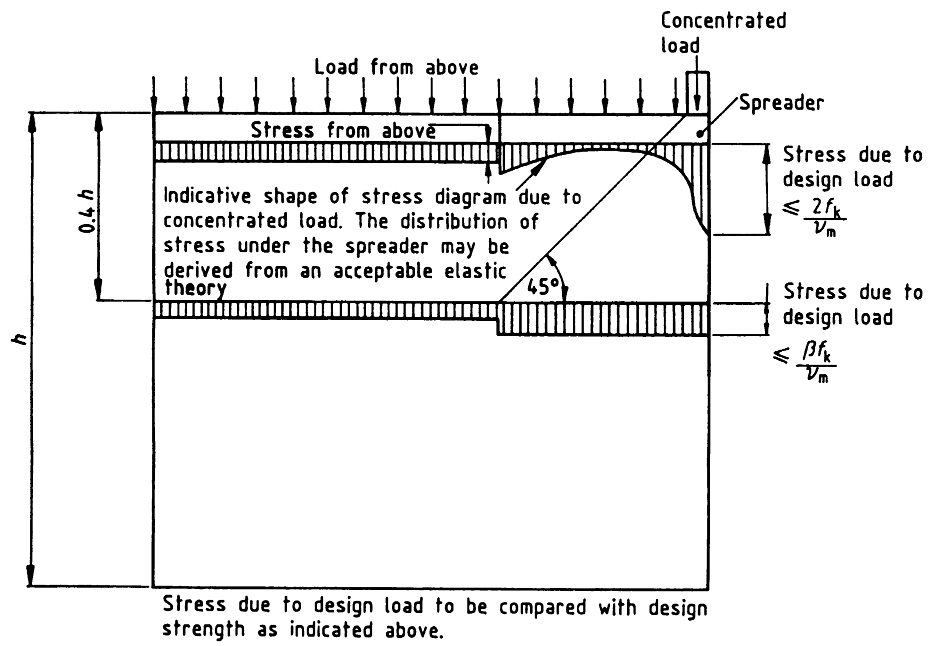
In this case, when checking the stress at a distance of $0.4h$ below the bearing, the design strength should be calculated in accordance with clause **32** [see Figure 6(b)].

35 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 **32** using clause **34** for concentrated loads and ignoring the effects of slenderness. The remainder of the wall should be designed in accordance with clause **32**.



(a) Load distribution for bearing types 1 and 2



(b) Load distribution for bearing type 3

Figure 6 — Concentrated loads: load distribution

36 Walls subjected to lateral load

36.1 General

Empirical guidance is given on certain wall shapes and conditions 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 **22**.

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 **5.6** of BS 8215:1998.

36.2 Support 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 7 and Figure 8.

Simple supports may be provided by:

a) ties complying with BS 1243, meeting the recommendations of Table 8 using the value of γ_m given in **27.5**;

b) ties meeting the design recommendations of DD 140-2 and tested in accordance with the methods given in DD 140-1, using the value of γ_m given in **27.5**.

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 ties or by the shear resistance of the masonry taking into account the damp proof course, if any. Where vertical twist ties conforming to BS 1243 are required to transmit compression, provided that any cavity gap between the wall and the supporting structure is not greater than 75 mm, their characteristic compressive resistance may be taken as:

a) 5.0 kN when laid in mortar designations (i) or (ii); or

b) 4.0 kN when laid in mortar designation (iii); or

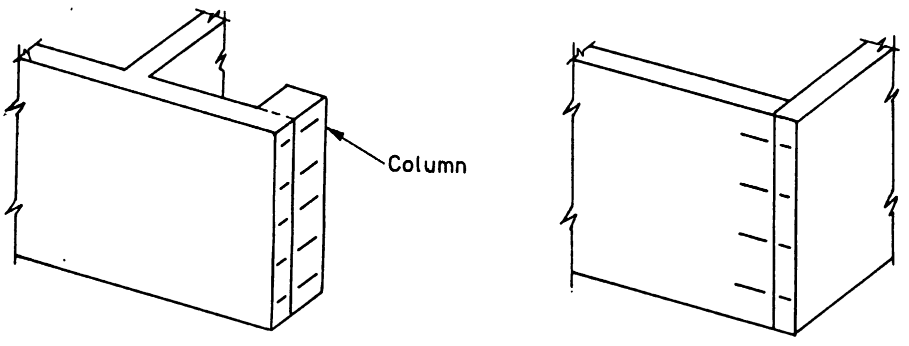
c) 2.5 kN when laid in mortar designation (iv).

For vertical twist ties conforming to BS 1243 in cavities up to 100 mm wide the characteristic compressive resistance of ties may be taken as:

a) 4.5 kN when laid in mortar designations (i) or (ii); or

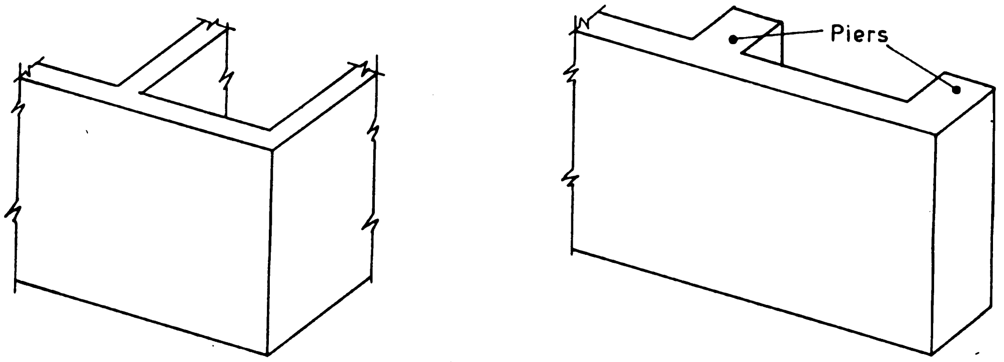
b) 3.5 kN when laid in mortar designation (iii); or

c) 2.0 kN when laid in mortar designation (iv).



(a) Metal ties to columns. Simple support; direct force restraint limited to values given in table 8.

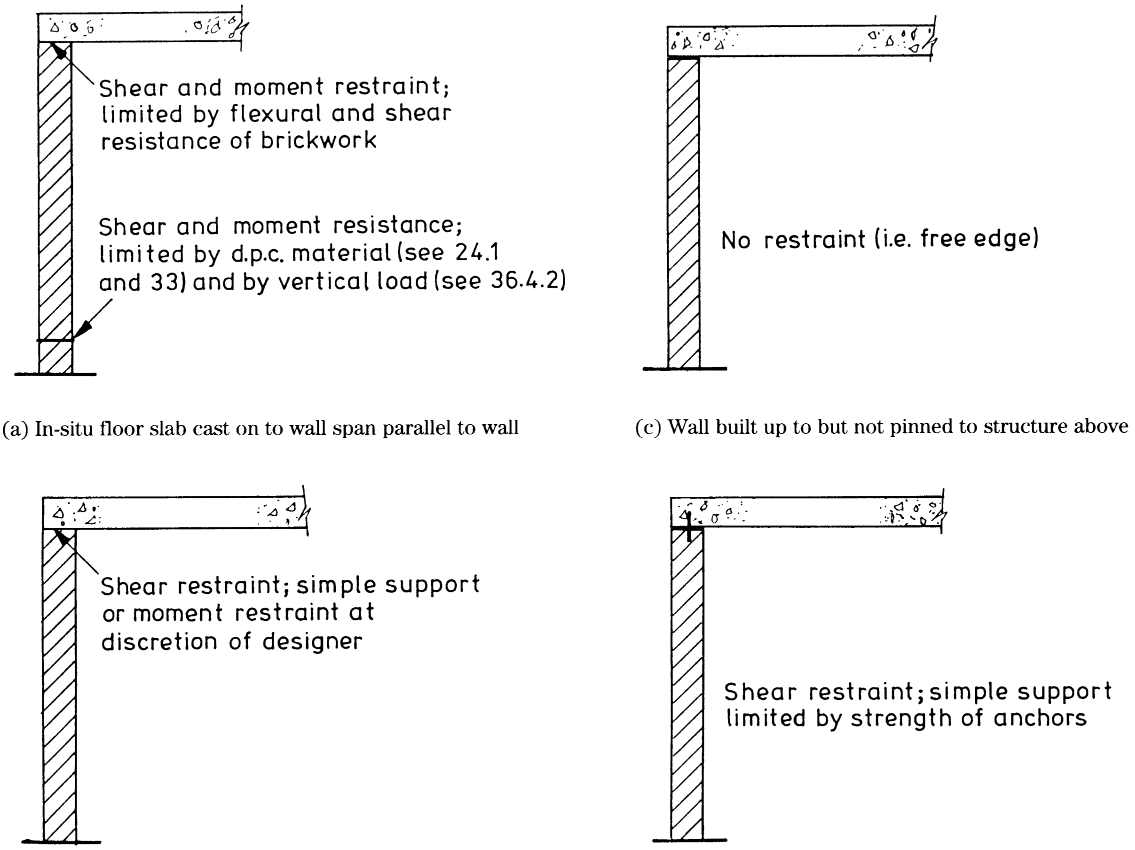
(c) Metal ties to columns or unbonded return walls. Shear and possibly moment restraint. Shear limited to values given in table 8.



(b) Bonded return walls. Restrained support; direct force and moment restraint limited by tensile strength of masonry as given in **24.1**.

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

Figure 7 — Vertical support conditions for laterally loaded panels



(a) In-situ floor slab cast on to 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 8 — Horizontal support conditions

When laid in mortar designations (i), (ii), (iii) or (iv), the characteristic compressive resistance of double triangle ties and of wire butterfly ties conforming to BS 1243 may be taken, respectively, as 1.25 kN and 0.5 kN for cavities up to 75 mm wide and 0.65 kN and 0.35 kN for cavities up to 100 mm wide, subject to the requirements detailed in **29.1.4** and **29.1.5**. The characteristic compressive resistance of ties not covered by the above may be determined in accordance with DD 140-1.

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 ties in accordance with Table 6. 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 ties to one leaf only, provided that there is adequate connection between the two leaves, particularly at the edges of the panels.

Table 8 — Characteristic strengths of wall ties used as panel supports

Type	Characteristic strengths of ties engaged in dovetail slots set in structural concrete			
	Tension			Shear
Dovetail slot types of ties	kN			kN
a) Galvanized or stainless steel fishtail anchors 3 mm thick, 17 mm min. width in 1.25 mm thick galvanized or stainless steel slot, 150 mm long, set in structural concrete	4.0			5.0
b) Galvanized or stainless steel fishtail anchors 2 mm thick, 17 mm min. width, in 2 mm thick galvanized or stainless steel slots 150 mm long, set in structural concrete	3.0			4.5
c) Copper fishtail anchors 3 mm thick, 17 mm min. width, in 1.25 mm copper slots, 150 mm long, set in structural concrete	3.5			4.0
	Characteristic loads in ties embedded in mortar			
	Tension			Shear[a]
	Mortar designations			Mortar designation
	(i) and (ii)	(iii)	(iv)	(i), (ii) or (iii)
Cavity wall ties[b]	kN	kN	kN	kN
a) Wire butterfly type:				
Zinc coated mild steel or stainless steel	3.0	2.5	2.0	2.0
b) Vertical twist type:				
Zinc coated mild steel or bronze or stainless steel	5.0	4.0	2.5	3.5
c) Double triangle type:				
Zinc coated mild steel or bronze or stainless steel	5.0	4.0	2.5	3.0

[a] Applicable only to cases where shear exists between closely abutting surfaces.
[b] See BS 1243:1978.

36.3 Limiting dimensions

In a laterally-loaded panel or freestanding wall built of masonry set in mortar designations (i) to (iv) and designed in accordance with clause **36**, 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) *Freestanding wall*

Height equal to 12t_{ef} or less.

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

36.4 Methods of design for laterally loaded wall panels

36.4.1 *General*

Masonry walls subjected to mainly lateral loads are not capable of precise design. There are, however, two approximate methods which 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 Appendix D.

36.4.2 *Calculation of design moments in panels*

Masonry walls are not isotropic and there is an orthogonal strength ratio, μ, (see **3.16**) depending on the brick or block and mortar used, as may be found from the characteristic flexural strengths given in clause **24**.

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 9;

γ_f is the partial safety factor for loads (clause **22**);

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 **27**, Table 4b)];

g_d is the design vertical dead load per unit area.

The bending moment coefficient, $\mu\alpha$, 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 **24.1**) is not exceeded.

Table 9 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:

$W_k \gamma_f h^2/8$

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.

36.4.3 ***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 **24**);

γ_m is the appropriate partial safety factor for materials [clause **27**, Table 4b)];

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.

36.4.4 ***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 an 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 **36.8**.

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:

$$t - t/10 - \delta$$

where

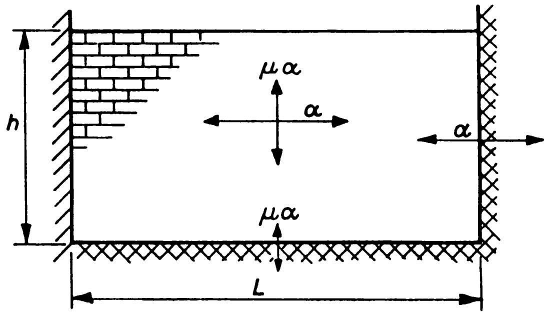
t is the overall thickness of the wall;

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

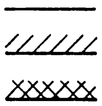
Table 9 — Bending moment coefficients in laterally loaded wall panels

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

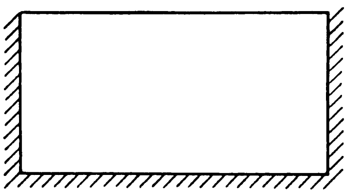
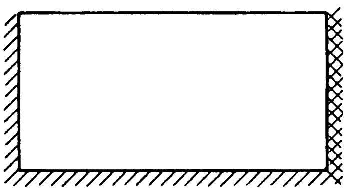
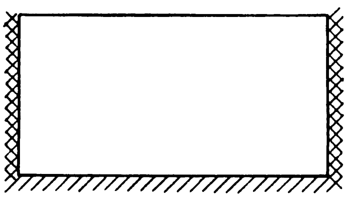
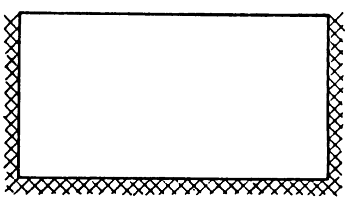
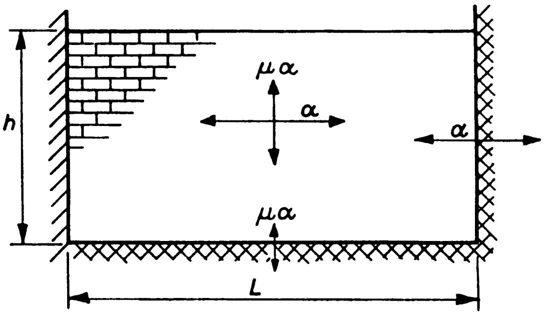
	μ	Values of α						
		h/L						
		0.30	**0.50**	**0.75**	**1.00**	**1.25**	**1.50**	**1.75**
 A	**1.00**	0.031	0.045	0.059	0.071	0.079	0.085	0.090
	0.90	0.032	0.047	0.061	0.073	0.081	0.087	0.092
	0.80	0.034	0.049	0.064	0.075	0.083	0.089	0.093
	0.70	0.035	0.051	0.066	0.077	0.085	0.091	0.095
	0.60	0.038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

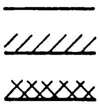
Table 9 — Bending moment coefficients in laterally loaded wall panels (*continued*)

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



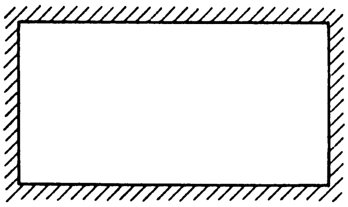
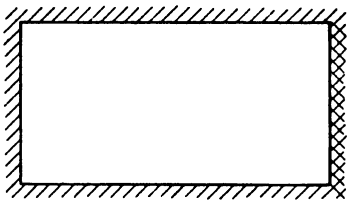
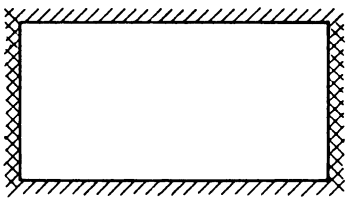
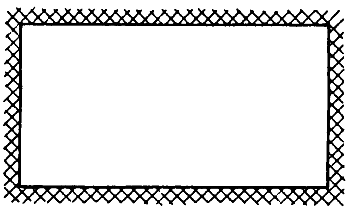
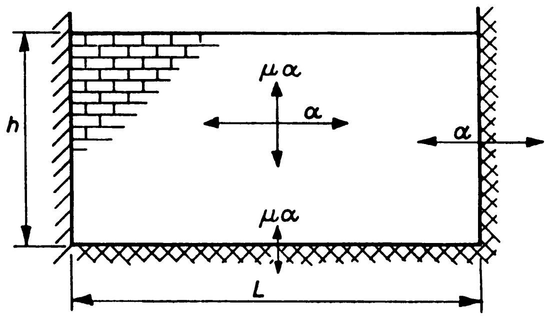
		Values of α						
	μ	h/L						
		0.30	**0.50**	**0.75**	**1.00**	**1.25**	**1.50**	**1.75**
 **E**	**1.00**	0.008	0.018	0.030	0.042	0.051	0.059	0.066
	0.90	0.009	0.019	0.032	0.044	0.054	0.062	0.068
	0.80	0.010	0.021	0.035	0.046	0.056	0.064	0.071
	0.70	0.011	0.023	0.037	0.049	0.059	0.067	0.073
	0.60	0.012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

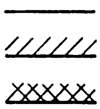
Table 9 — Bending moment coefficients in laterally loaded wall panels (*continued*)

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



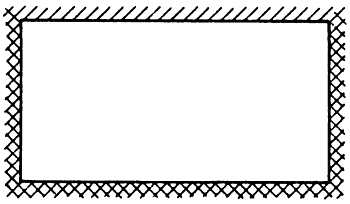
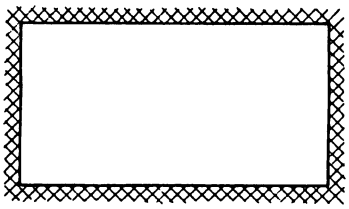
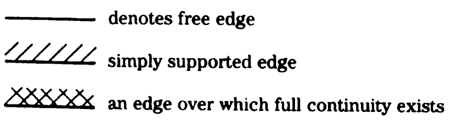
Panel	μ	Values of α, h/L = 0.30	0.50	0.75	1.00	1.25	1.50	1.75
 H	**1.00**	0.005	0.011	0.018	0.024	0.029	0.033	0.036
	0.90	0.006	0.012	0.019	0.025	0.030	0.034	0.037
	0.80	0.006	0.013	0.020	0.027	0.032	0.035	0.038
	0.70	0.007	0.014	0.022	0.028	0.033	0.037	0.040
	0.60	0.008	0.015	0.024	0.030	0.035	0.038	0.041
	0.50	0.009	0.017	0.025	0.032	0.036	0.040	0.043
	0.40	0.010	0.019	0.028	0.034	0.039	0.042	0.045
	0.35	0.011	0.021	0.029	0.036	0.040	0.043	0.046
	0.30	0.013	0.022	0.031	0.037	0.041	0.044	0.047
 I	**1.00**	0.004	0.009	0.015	0.021	0.026	0.030	0.033
	0.90	0.004	0.010	0.016	0.022	0.027	0.031	0.034
	0.80	0.005	0.010	0.017	0.023	0.028	0.032	0.035
	0.70	0.005	0.011	0.019	0.025	0.030	0.033	0.037
	0.60	0.006	0.013	0.020	0.026	0.031	0.035	0.038
	0.50	0.007	0.014	0.022	0.028	0.033	0.037	0.040
	0.40	0.008	0.016	0.024	0.031	0.035	0.039	0.042
	0.35	0.009	0.017	0.026	0.032	0.037	0.040	0.043
	0.30	0.010	0.019	0.028	0.034	0.038	0.042	0.044

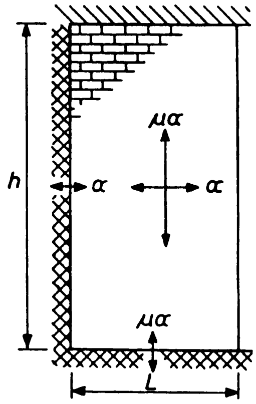
Table 9 — Bending moment coefficients in laterally loaded wall panels (*concluded*)

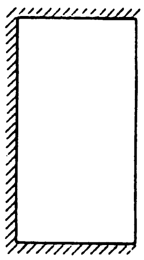
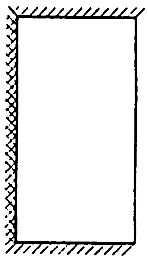
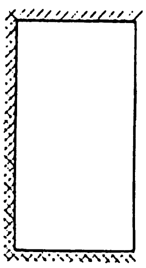
NOTE 1 Linear interpolation of μ and h/L is permitted.

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





	μ	Values of α						
		h/L						
		0.30	**0.50**	**0.75**	**1.00**	**1.25**	**1.50**	**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

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 structural 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 [clause **27**, Table 4a)].

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

36.4.5 ***Design lateral strength for cavity walls***

The design lateral strength for a cavity wall tied with vertical twist-type wall ties or ties with equivalent strength and stiffness should 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 butterfly or double triangle ties are to be used the design lateral strength of the cavity wall may be taken as the sum of the design lateral strengths of the two leaves, provided that the ties are capable of transmitting the compressive forces to which they are subjected; when the ties are not capable of transmitting the full force, the contribution of the appropriate leaf should be limited accordingly.

36.5 Method of design for freestanding walls

36.5.1 ***General***

Freestanding 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 **36.4.3** and **36.4.4** respectively. The piers should then be designed as cantilevers to resist the reaction from the panel. Mortar should not be weaker than mortar designation (iii) (see Table 1).

36.5.2 ***Calculation of design moment in freestanding walls***

The design moment of a freestanding 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 (clause **21**);

γ_f is the partial safety factor for loads (clause **22**);

h is the clear height of the wall or pier above restraint;

Q_k is the characteristic imposed load (clause **21**);

h_L is the vertical distance between the point of application of the horizontal load Q_k and the lateral restraint.

36.5.3 ***Calculation of design moment of resistance of freestanding walls***

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

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

where

- f_{kx} is the characteristic flexural strength at the critical section, which may be the damp proof course (clause **24**);
- γ_m is the appropriate partial safety factor for materials [clause **27**, Table 4b)];
- 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 **36.4.3**);
- 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 **24.1**) a freestanding 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 **23**).
- γ_m is the appropriate partial safety factor for materials [clause **27**, Table 4a)].

36.6 Retaining walls

Where retaining walls are to resist lateral earth and water pressure only, they should be designed in accordance with clauses **21** and **22**.

36.7 Foundation walls

Where a wall carries vertical loading combined with lateral loading the earth and water pressure should be treated as detailed in **36.6**.

36.8 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 **32** and Appendix B or from the relationship:

$$q_{lat} = \frac{4tn}{h_a^{\,2}}$$

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 **22**d)];
- 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, the design load is not less than 0.1 N/mm², 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 10 in which h_a is the clear height of the wall or column L is the length of the wall or column.

Table 10 — 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

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

36.9.1 *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 conditions shown below.

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 (clause **24** and clause **27**). 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 the wind load.

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

36.9.2 *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 **21** and relevant partial safety factors for loads given in clause **22** 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.

36.9.3 ***Calculations of design moment of resistance***

36.9.3.1 *Condition a) of* ***36.9.1***

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 [clause **27**, Table 4b)];
- 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.

36.9.3.2 *Condition b) of* ***36.9.1***

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

36.9.4 ***Integrity of sections in shear***

36.9.4.1 *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 either 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 **25**.

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

$$ru = 12t_w sv/(0.87f_y)$$

where

- r is the width of the connector;
- u is the thickness of the connector;
- t_w 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_y is the characteristic tensile strength of the connector.

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

36.9.4.2 *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 **25**.

Section 5. Design: accidental damage

37 Design: accidental damage

37.1 General guidance

The general recommendations in **20.3** are aimed at the limitation of accidental damage and preservation of structural integrity and are applicable to all building categories. No additional detailed recommendations are made in respect of category 1 buildings (four storeys and below) but for category 2 buildings (five storeys and above), assessment of residual stability and spread of damage, following the removal of a loadbearing element, as defined in Table 11, or alternatively, provision within the structure of vertical or horizontal tying, or both, is recommended.

The options available to the designer are given in Table 12.

If option (1) is adopted, the structure should be examined for the effect of the removal, within each storey, bay, span or cantilever, of any single vertical or horizontal loadbearing element, as appropriate, unless it is designed as a protected member. Similar analysis, in respect of vertical elements only, is recommended under option (2), but in the case of option (3) 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.

37.1.1 *Protected member*

A protected member is a member which, together with its essential supports, can withstand, without collapse, its reduced design load in accordance with clause **22**d), and an accidental design load of 34 kN/m^2 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 applied in the direction under consideration, or such lesser reaction as might reasonably be transmitted having 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 **36.8** but using the formula:

$$q_{\text{lat}} = \frac{7.6tn}{h_a^{\ 2}}$$

NOTE The formula incorporates a safety factor of 1.05.

Table 11 — Loadbearing elements

Type of loadbearing element	Extent
Beam	Clear span between supports or between a support and the extremity of a member
Column	Clear height between horizontal lateral supports
Slab or other floor and roof construction	Clear span between supports and/or temporary supports or between a support and the extremity of a member
Wall incorporating one or more lateral supports (note 2)	Length between lateral supports or length between a 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 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 a), b) and c) of **37.5** or purpose-designed structural elements.

37.2 Partial safety factors

For consideration of misuse and accident, the partial safety factors for loads should be taken from clause **22**d) and those for material strength γ_m from **27.3**, **27.4**, **27.5** and **37.1.1**.

37.3 Horizontal ties

The requirements for peripheral, internal and column or wall ties can be obtained from Table 13.

Peripheral, internal and column or wall ties [options (2) and (3) in Table 12] should be provided at each floor level and at roof level, but where the roof is of lightweight[2)] construction no such ties need be provided at that level.

Horizontal ties may be provided in whole or in part by structural members which may already be fully stressed in serving other purposes, e.g. reinforcement in a concrete floor slab or masonry in tension. Ties should be positioned to resist most effectively accidental damage.

37.4 Vertical ties

The requirements for full vertical ties can be obtained from Table 14.

For full vertical tying to be considered effective [option (3)], 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 kilonewtons per metre width, where F_t is as given in Table 13. 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.

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 **37.1.1**.

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.

37.5 Loadbearing elements

For the purposes of this clause the definition of "loadbearing elements" is as given in Table 11, where h is the clear height of wall between horizontal lateral supports.

Lateral supports in relation to wall elements as defined in Table 11 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 kilonewtons per metre height of wall (Table 13), 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 $1.5F_t$ kilonewtons per metre height of wall (Table 13).

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 $0.5F_t$ kilonewtons per metre height of wall (Table 13).

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

[2)] Roofs comprising timber or steel trusses, flat timber roofs or roofs incorporating concrete or steel purlins with asbestos or wood-wool deck may be regarded as lightweight.

Table 12 — Detailed accidental damage recommendations

<table>
<tr><td>Category 1
All buildings of four storeys and below</td><td colspan="4">Plan form and construction to provide robustness, interaction of components and containment of spread of damage (see clause 20)</td></tr>
<tr><td rowspan="3">Category 2
All buildings of five storeys and above</td><td rowspan="3">Plan form and construction to provide robustness, interaction of components and containment of spread of damage (see clause 20)</td><td colspan="3">Additional detailed recommendations for category 2</td></tr>
<tr><td>Option (1)</td><td>Option (2)</td><td>Option (3)</td></tr>
<tr><td>Vertical and horizontal elements, unless protected, proved removable, one at a time, without causing collapse</td><td>Horizontal ties
Peripheral, internal and column or wall in accordance with 37.3 and Table 13
Vertical ties
None or ineffective
Vertical elements, unless protected, proved removable, one at a time, without causing collapse</td><td>Horizontal ties
Peripheral, internal and column or wall in accordance with 37.3 and Table 13
Vertical ties
In accordance with 37.4 and Table 14</td></tr>
</table>

Table 13 — Requirements for full peripheral, internal and column or wall ties

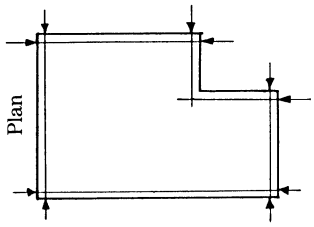
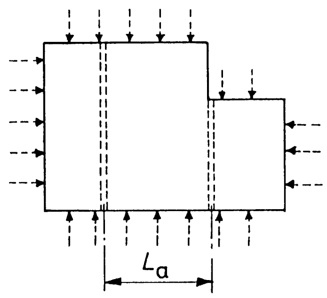
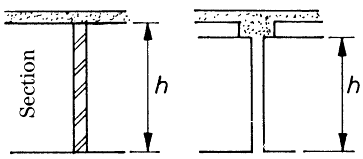
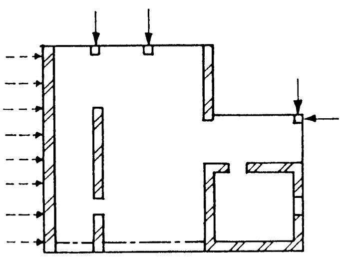
Basic horizontal tie force = F_t = 60 kN or 20 + 4N_s kN, whichever is the lesser of the two values; N_s is the number of storeys (including ground and basement).				
Type of tie	**Unit of tie force**	**Size of design tie force**	**Location of tie force (arrowed)**	**Fixing requirements and notes**
A. *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 width	F_t or $\frac{F_t\,(G_k + Q_k)}{7.5} \times \frac{L_a}{5}$ whichever is the greater 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)   Flat slab Beam and slab	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/m²; 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.
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; or 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 14 — Requirements for full vertical ties

Minimum thickness of a solid wall or one loadbearing leaf of a cavity wall	150 mm
Minimum characteristic compressive strength of masonry	5 N/mm²
Maximum ratio h_a/t	20
Allowable mortar designations (see Table 1)	(i), (ii), (iii)
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 mm² 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.	

Appendix A Testing

A.1 Mortar testing

A.1.1 *Preliminary tests*

At least 6 weeks before the masonry building 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 specimens of one of the following types:

a) 70.7 mm cube

b) 100 mm cube

c) 100 mm × 25 mm × 25 mm prisms

d) 160 mm × 40 mm × 40 mm prisms

made with mortar of a consistency corresponding to a 10 mm penetration of the dropping ball, should be made by the "without suction" method, cured hydraulically and tested for compressive strength, in accordance with the procedures given in BS 4551. The type of specimen to be later used on the site should be identical with that used for these preliminary tests.

A.1.2 *Interpretation of test results*

The average compressive strengths for the various mortar designations are shown in Table 1.

If desired, half of the test specimens may be tested at 7 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 additive 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 then 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.1.3 *Site tests*

Six specimens of one of the following types:

a) 70.7 mm cube

b) 100 mm cube

c) 100 mm × 25 mm × 25 mm prisms

d) 160 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 4551.

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.

A.2 Experimental determination of characteristic compressive strength of masonry

A.2.1 *General*

A characteristic compressive strength for masonry, f_k, can be obtained from the ultimate strength of a brickwork or blockwork panel tested to destruction in accordance with this appendix. It should be noted that modifications to the test results may be necessary, depending on the compressive strength of the structural units.

A.2.2 ***Type of test panel***

Duplicate tests should be carried out on nominally identical panels. These should be from 1.2 m to 1.8 m in length with a minimum cross-sectional area of 0.125 m^2 and from 2.4 m to 2.7 m in height. Where, in special cases, it is required to test panels having dimensions outside these limits, the general principles laid down in this test procedure should be applied and the details should be reported. The construction and bond of the test wall should correspond to those to be used in practice.

A.2.3 ***Method of testing***

The load should be applied uniformly over the whole area of the top and bottom of the panel. The platens or cross-heads through which the load is applied should be restrained against rotation to produce a flat-ended condition. The load should be applied at a rate of 1 N/mm^2 per minute.

A.2.4 ***Structural units for test panels***

The bricks and blocks used in the construction of the test panels should be representative of those to be used in actual structures. The compressive strength, p_u, of a sample of the structural units should be obtained using the test methods given in BS 187, BS 3921 or BS 6073-1.

Where the structural units are not subjected to regular quality control, the manufacturer, prior to the tests, should state a "specified compressive strength", p_o, below which samples for future consignments supplied for the structure would not be expected to fall. Then the "unit reduction factor", ψ_u, should be calculated as follows:

$$\psi_u = \frac{p_o}{p_u} \text{ with a maximum of 1.0.}$$

Where the structural units are from a site in which the special category of manufacturing control is in operation, the manufacturer will specify an "acceptance limit" for compressive strength, p_{lim}, for the structural units (see **27.2**). Then the "unit reduction factor", ψ_u, should be calculated as follows:

$$\psi_u = \frac{p_{lim}}{p_u} \text{ with a maximum of 1.0.}$$

A.2.5 ***Mortar for test panels***

The sand for the mortar used in the construction of the test panels should be representative of that to be used in actual structures.

The mortar used in the construction of the test panels should consist of weighed amounts of dry materials determined from the volume proportions given in Table 1, corresponding to the appropriate designation. The amount of water used for the first batch should be that needed for workability and this amount should not be changed for subsequent batches.

Samples of mortar should be taken from the mixing board to make mortar specimens and the simulated site strength, determined in accordance with **A.1.3**, should be reported.

A.2.6 ***Curing and age of testing***

The test panels should be covered with polyethylene sheets for a period of three days after construction and then left uncovered until tested. It is recommended that the panels be tested at an age of 28 days, and this may be extended to 35 days when circumstances require this extension. The tests may be made earlier by agreement.

The compressive strength of the mortar should be determined at the age at which the panels are tested.

A.2.7 ***Calculation of characteristic compressive strength,*** f_k

The characteristic compressive strength, f_k, for the masonry under consideration may be calculated as follows:

$$f_k = \frac{F_m}{A} \times \frac{\psi_u \psi_m}{1.2}$$

where

F_m is the mean of the maximum loads carried by the two test panels;

A is the cross-sectional area of each panel;

ψ_m is the reduction factor for strength of mortar given in Table 15;

ψ_u is the unit reduction factor for sample structural units.

The factor 1.2 is introduced to relate the characteristic value to the mean value.

Where the height of wall between flat ends exceeds 20 times the thickness, it is possible that slenderness effects may significantly reduce the test results from those that would be expected from a wall in the recommended height range. This effect can be approximately assessed by measuring the lateral deflection of the wall at mid-height under test.

If y_u is the deflection measured just before the point when the ultimate load is attained, it will be appropriate to increase the test value of F_m/A by $(t/(t–y_u))$, where t is the thickness of the wall. The increase, however, should not be greater than 15 %.

Table 15 — Reduction factor, ψ_m, for mortar strength

Ratio = $\frac{\text{Strength of mortar used for test panels}}{\text{Specified minimum site compressive strength}}$	Reduction factor, ψ_m
1.0 to 1.5	1.00
2.0	0.93
2.5	0.88
3.0	0.84
3.5	0.81
4.0	0.78
NOTE Linear interpolation between values is permitted.	

A.3 Determination of characteristic flexural strength of masonry

A.3.1 *Test procedure*

Characteristic flexural strengths for the two orthogonal directions may be determined by testing wallettes built of masonry units and mortars representative of those to be used on site.

Blocks should be immersed in water for 5 min to 6 min and allowed to drain prior to building the wallettes. The wallettes should be built within 1 h of the blocks being removed from the water. Bricks having a suction rate of more than 1.5 kg/(m²·min) may be docked or the water retentivity of the mortar may be adjusted. The method of conditioning the bricks should be recorded in the test report.

Immediately after building, each wallette should be pre-compressed with three courses of bricks laid dry, or by the equivalent uniformly distributed mass. The wallette should then be cured[3] and maintained undistributed until testing.

The wallette should be tested in the vertical attitude under four-point loading (see Figure 9). The equipment should accommodate variations of plane. Suitable precautions should be used at the contact area of the bearings to ensure that contact is provided over the full width of the masonry, e.g. a hollow rubber bolster of at least 7 mm wall thickness and 10 mm bore containing 8 mm diameter steel rod or a hydraulic bolster consisting of 18 mm o.d. polyester-reinforced PVC hose filled with water and sealed.

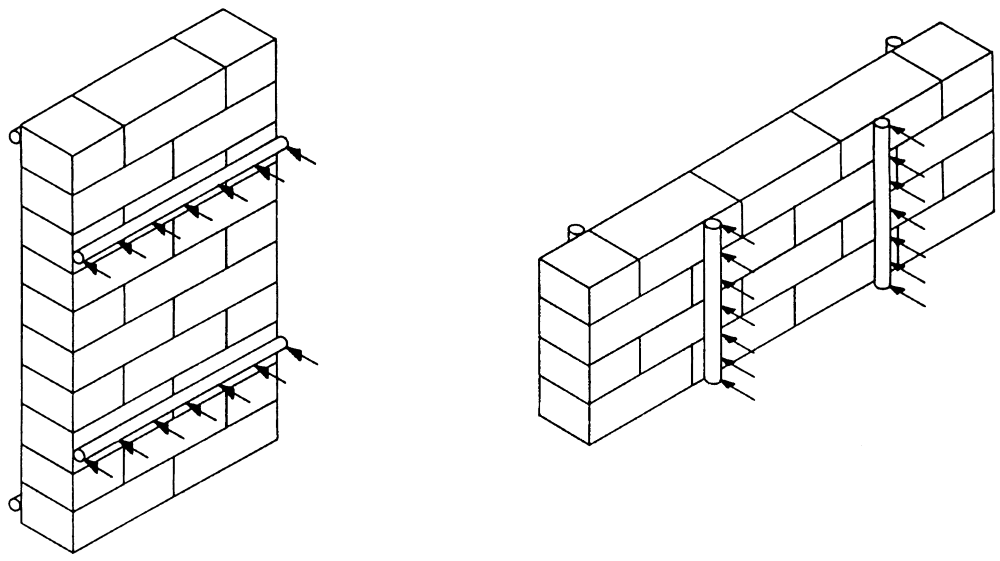
The outer bearings should be about 50 mm from the edge of the wallette. The spacings of the inner bearings may be varied to suit the format of the masonry but should be between 0.4 and 0.6 times the spacing of the outer bearings. The inner bearings should be located so that they are, as far as practicable, midway between the nearest mortar joints which are parallel to the bearings.

The base of each wallette should be free from frictional restraint, e.g. by setting it on two layers of polytetrafluoroethylene (PTFE) or on ball, needle or roller bearings.

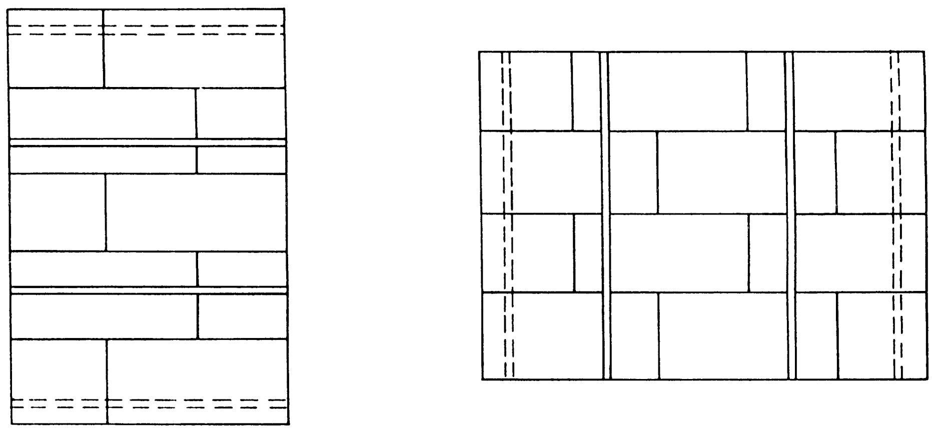
The rate of increase in flexural stress should be between 0.3 and 0.4 N/(mm²/min).

The replicates should be tested in each format at an age of 28 ± 1 days.

[3] Generally, close covering until testing with a material that does not permit water vapour penetration is most satisfactory; this was the method used to derive the figures in Table 3.



(a) Brick wallettes



(b) Block wallettes

Figure 9 — Testing of wallettes

A.3.2 ***Format of wallettes***

For brickwork not more than 102.5 mm thick, the format of wallette where the surface of failure will be at right angles to the bed joints should be four bricks long by four courses high. Where the surface of failure will be parallel to the bed joints, the format should be two bricks long by 10 courses high.

For blockwork, the format of wallette where the surface of failure will be at right angles to the bed joints should be 2½ blocks long by 4 block courses high. Where the surface of failure will be parallel to the bed joints, the format should be 1½ blocks long by 5 block courses high.

A.3.3 ***Calculation of characteristic flexural strength from experimental results***

Any failure other than one occurring between the inner bearings should be discounted in calculations. Not less than eight results should be available when calculating the characteristic flexural strength.

If the flexural strengths obtained from the replicates are:

$x_1, x_2, x_3 \ldots x_v$, the values

$y_1, y_2, y_3 \ldots y_v$, should be calculated

where

v = number of replicates;

y = log x.

The mean $(\bar{y})$ and standard deviation (s) of y should then be calculated using the following formulae:

$$\bar{y} = (y_1 + y_2 + y_3 + \ldots + y_v)/v$$

$$s = \sqrt{\frac{\left(y_1^2 + y_2^2 + \ldots + y_v^2\right) - \left[\left(y_1 + y_2 + \ldots + y_v^2\right)/v\right]}{v - 1}}$$

Then $y_c = \bar{y} - \kappa_s$.

where κ =

1.922 for 10 results;

1.961 for 9 results;

2.010 for 8 results.

Characteristic flexural strength = antilog (y_c).

Appendix B
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 **31**, 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:

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

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:

$$e_t = 0.6e_x + e_a \quad (2)$$

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 Assumptions 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, of greater than $0.05t$, the eccentric load should be assumed to be resisted by a rectangular stress block with a constant stress of $1.1f_k/\gamma_m$ (see Figure 10). It follows that the design vertical load capacity of the member is:

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

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 **23**;

γ_m is the appropriate partial safety factor for materials as defined in [clause **27**, Table 4a)].

Comparing the expressions for capacity given in **32.2** with equation (3), it will be seen that:

$$\beta = 1.1\ [1 - (2e_m/t)] \quad (4)$$

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

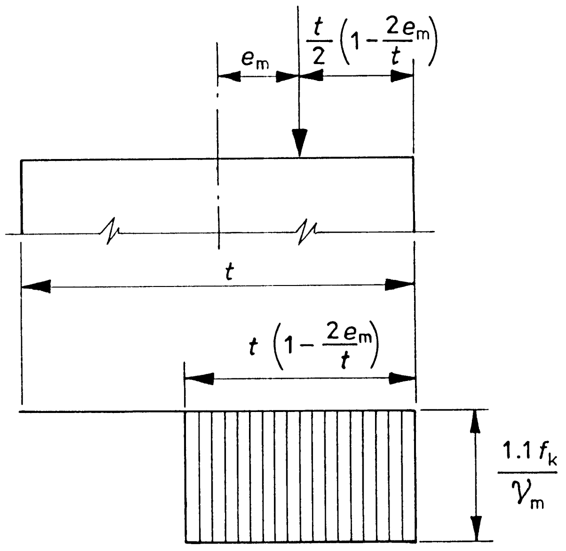


Figure 10 — Stress block under ultimate conditions

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 **32**.

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 **32**.

Appendix C
Connections to floors and roofs by means of metal anchors and joist hangers capable of resisting lateral movement

Figure 11 to Figure 25 illustrate connections which may be used to provide lateral restraint in accordance with **28.2.2**. In the illustrations, floors are generally shown; however, the same details are applicable to roofs. The effective cross-section of anchors and of their fixings should be capable of resisting the loads as specified in **28.2.1**, assuming a stress equal to the characteristic yield strength (or its equivalent) as laid down in the appropriate British Standard divided by γ_m = 1.15. Anchors 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. Galvanized mild steel anchors having a cross-section of 30 mm × 5 mm may be assumed to have adequate strength in buildings of up to six storeys in height.

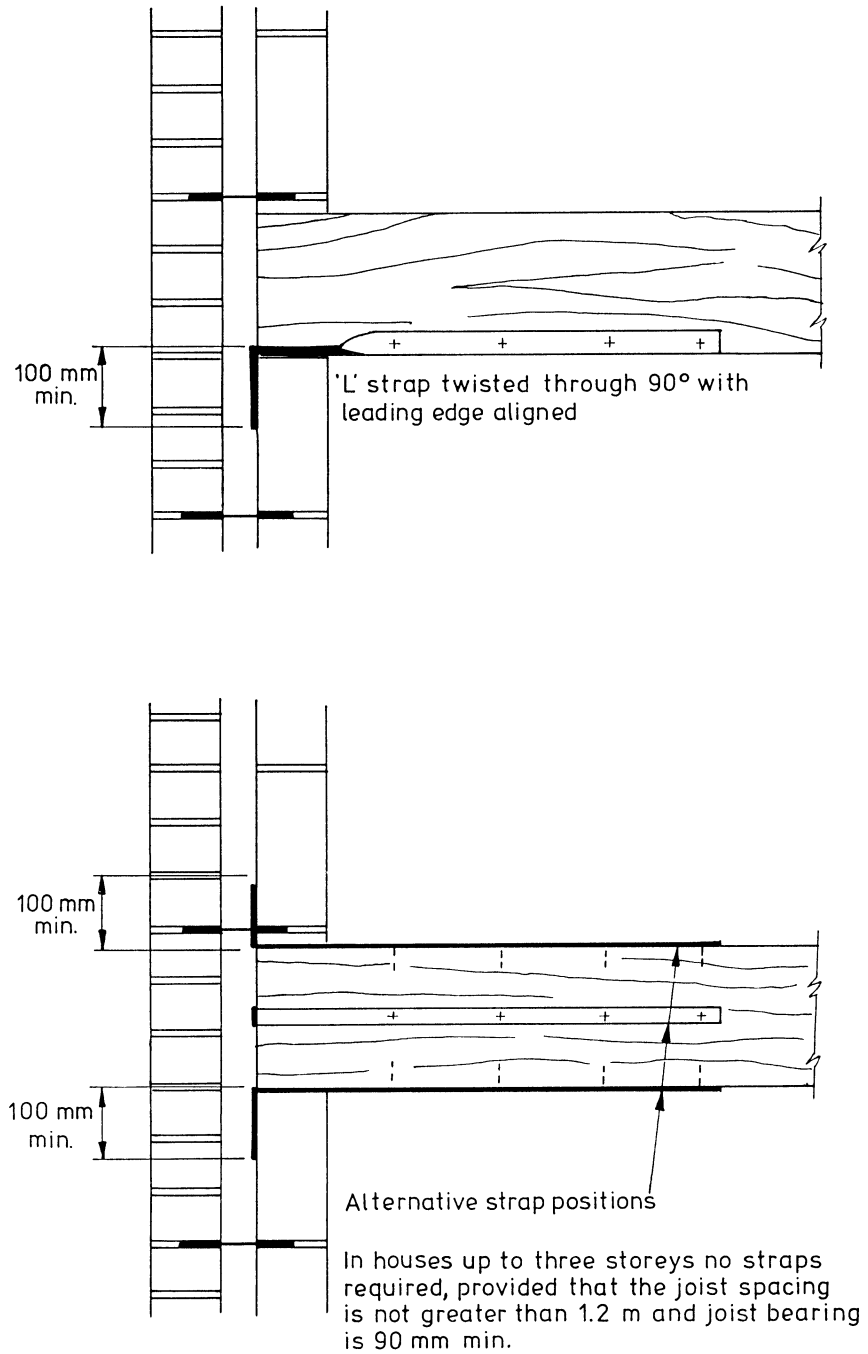


Figure 11 — Timber floor bearing directly on to wall

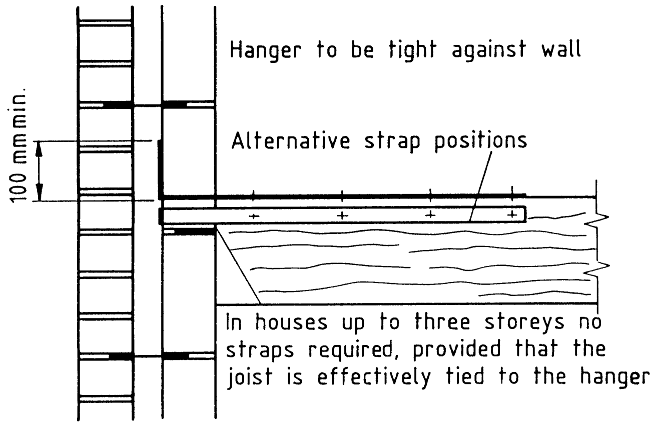


Figure 12 — Timber floor using typical joist hanger

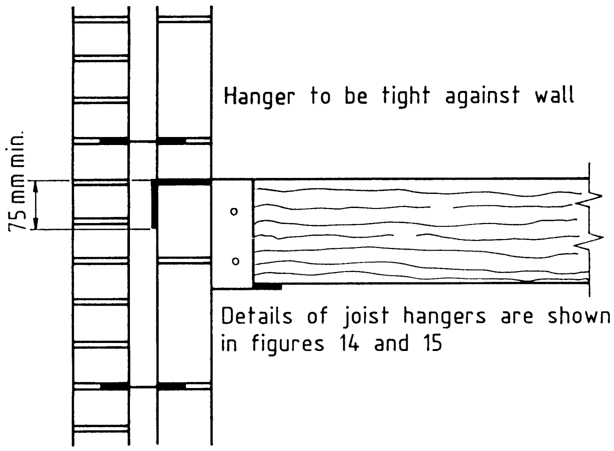


Figure 13 — Timber floor using nailed or bolted joist hangers acting as tie

All dimensions in millimetres

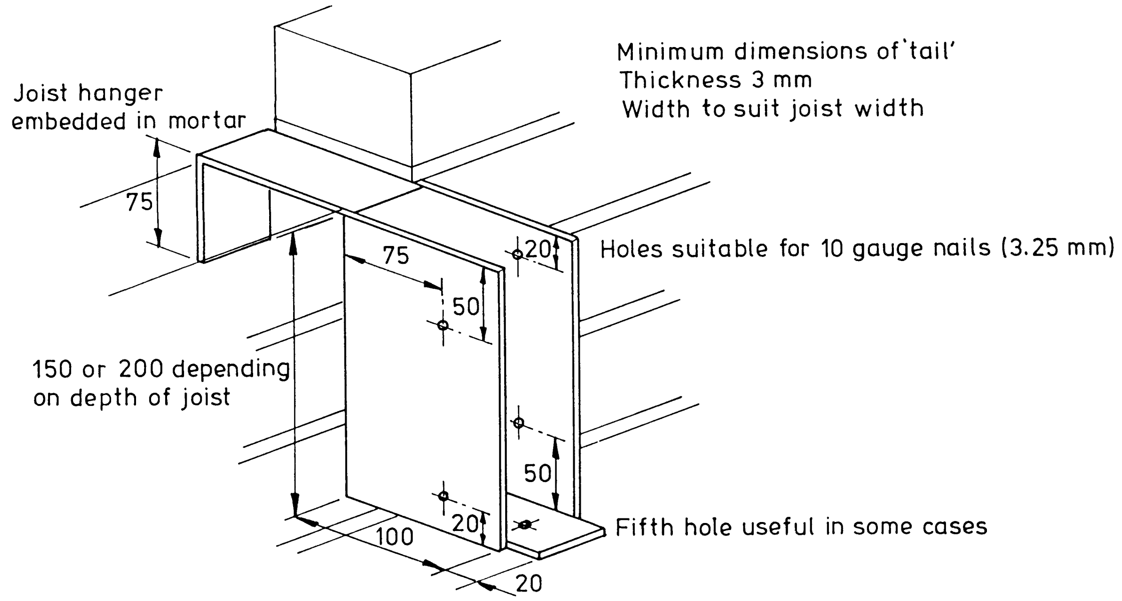


Figure 14 — Joist hanger as tie: nailed form

All dimensions in millimetres

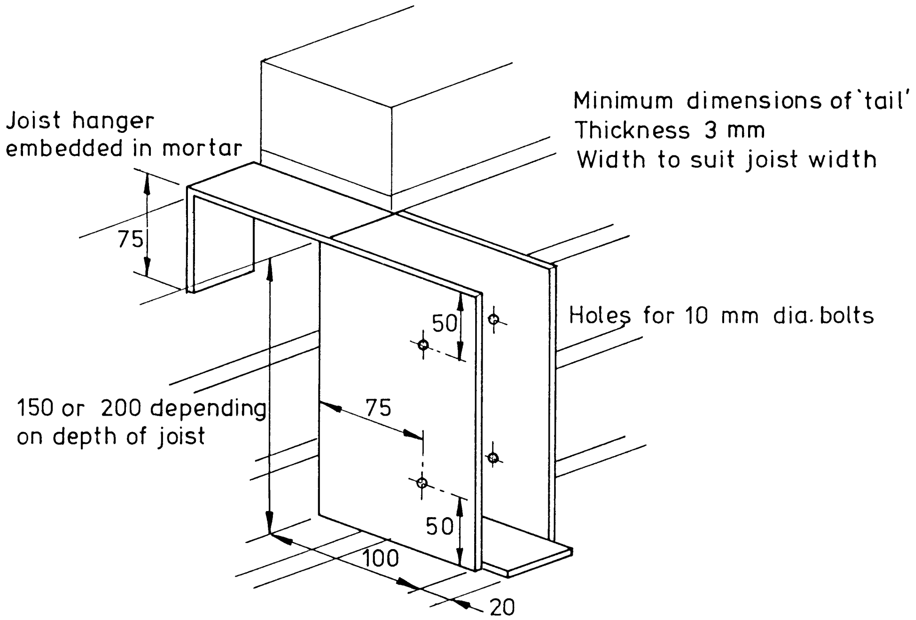


Figure 15 — Joist hanger as tie: bolted form

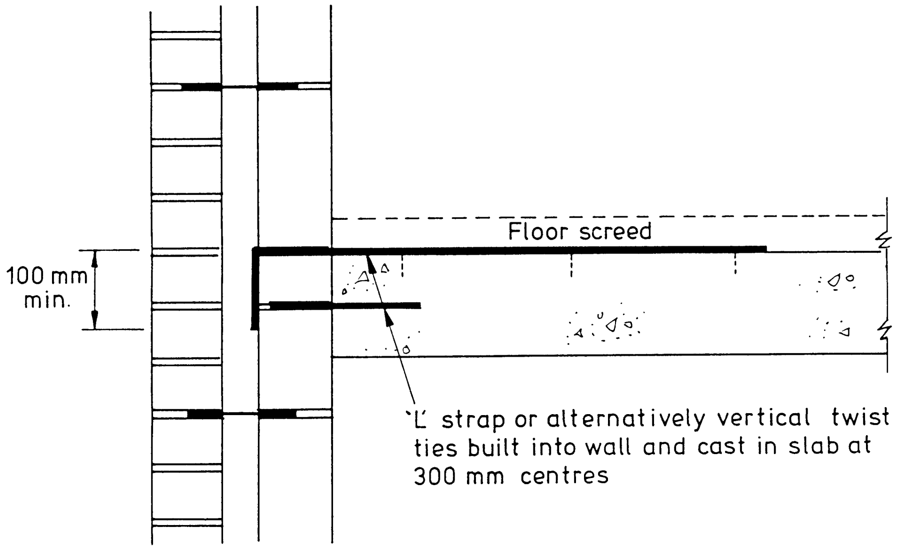


Figure 16 — In-situ concrete floor abutting external cavity wall

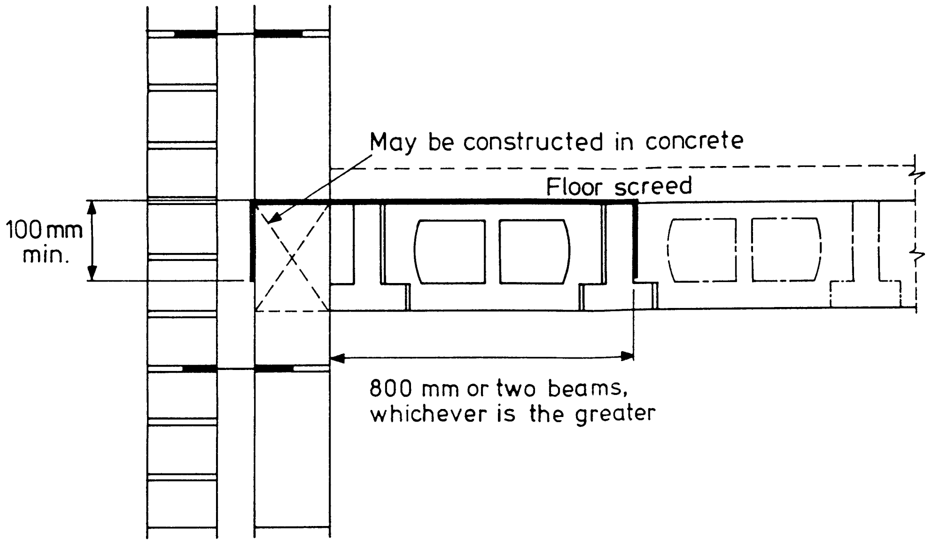


Figure 17 — Beam and pot floor abutting external cavity wall

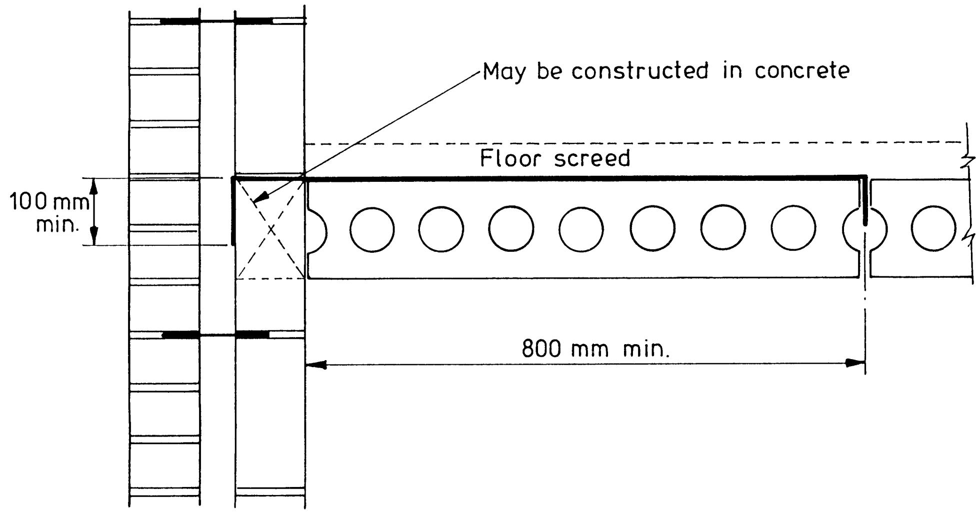


Figure 18 — Precast units abutting external cavity wall

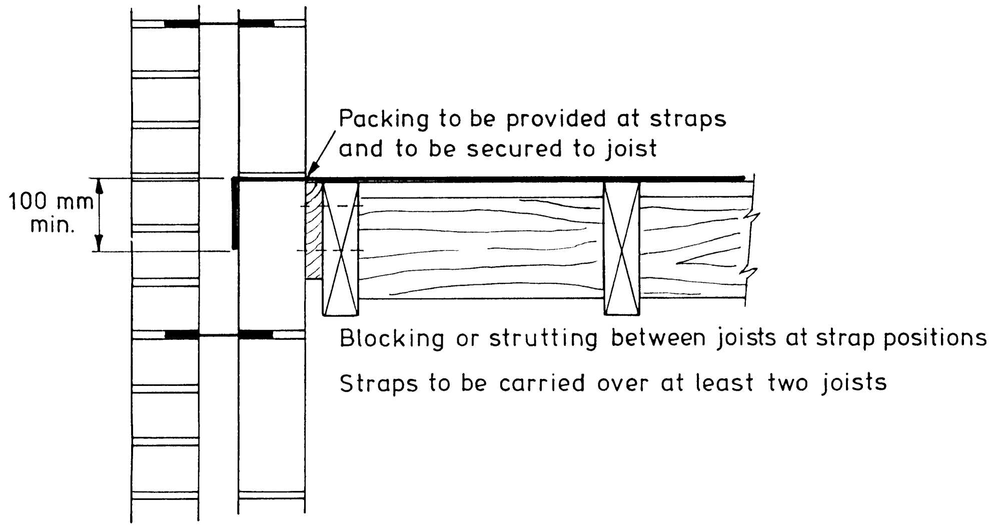


Figure 19 — Timber floor abutting external cavity wall

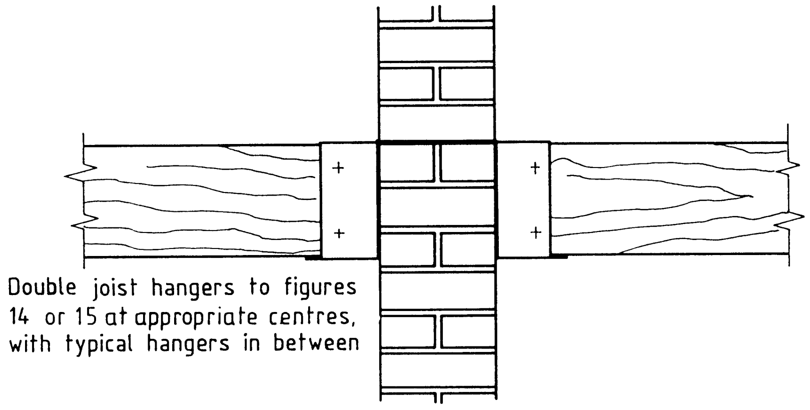


Figure 20 — Timber floor using double joist hanger acting as tie

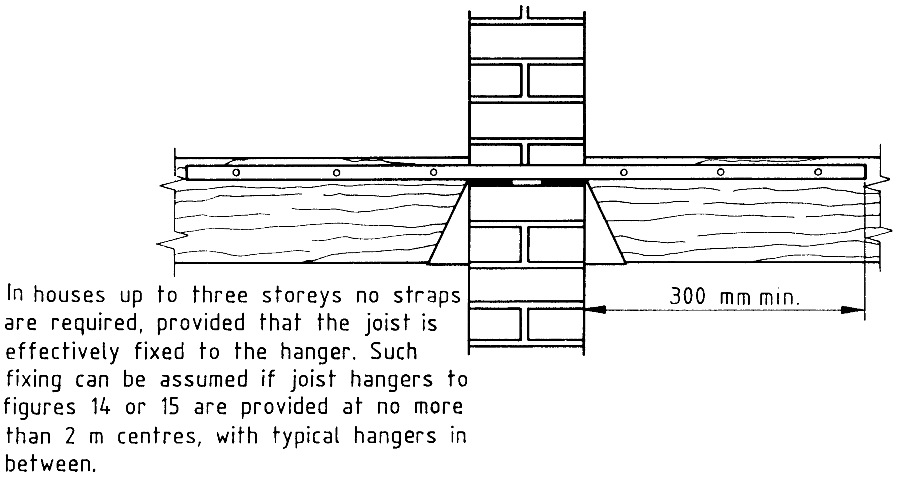


Figure 21 — Timber floor using typical joist hanger

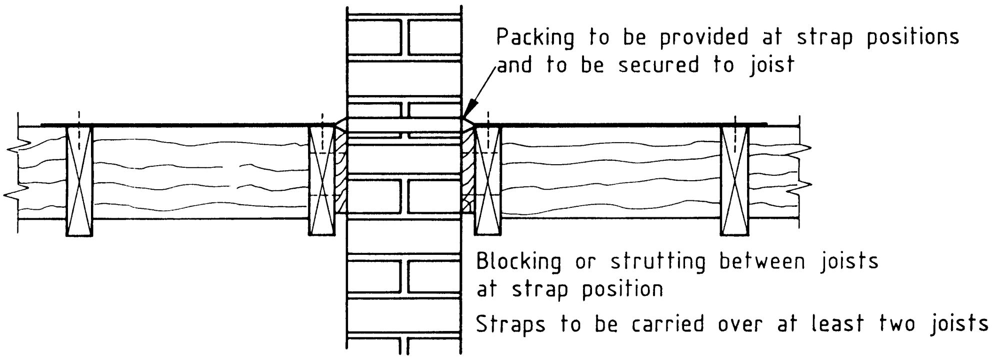


Figure 22 — Timber floor abutting internal wall

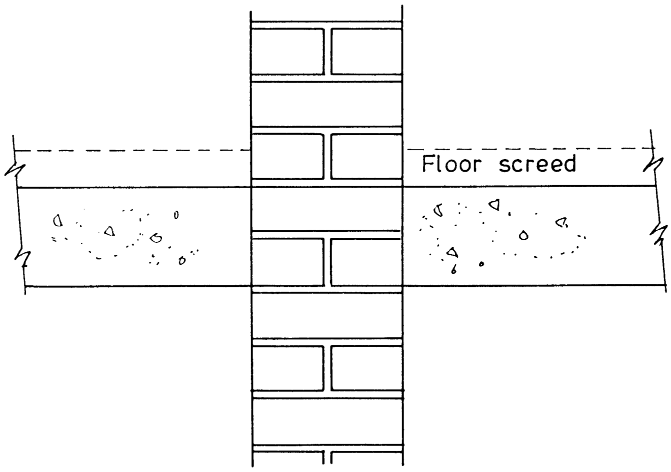


Figure 23 — In-situ floor abutting internal wall

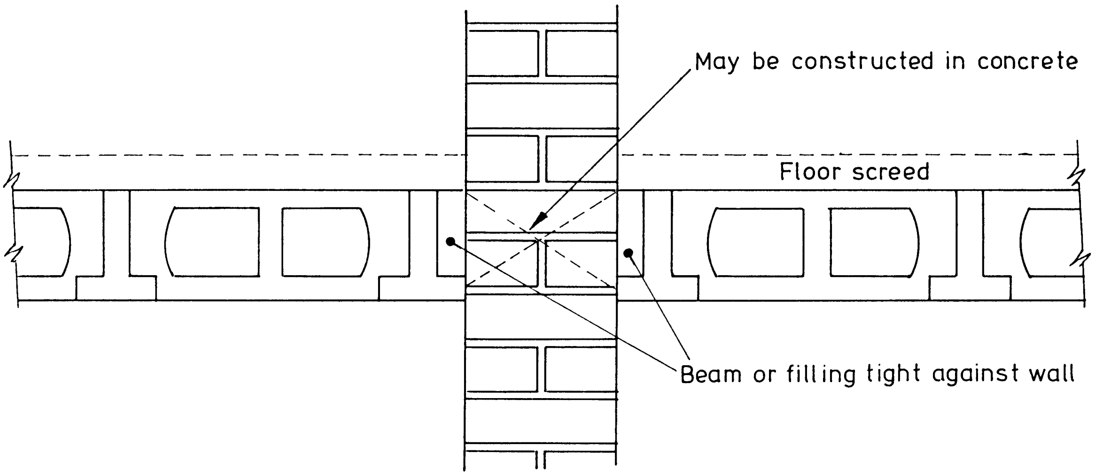


Figure 24 — Beam and pot floor abutting internal wall

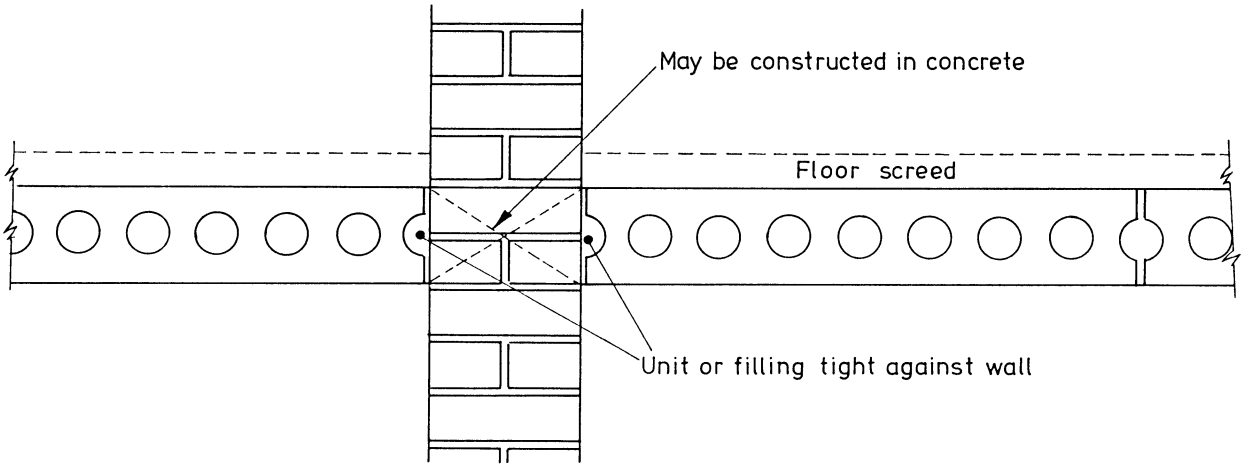


Figure 25 — Precast units abutting internal wall

Appendix D
Laterally loaded panels of irregular shape, or those containing openings

The guidance given in clause **36** on the design of laterally loaded panels without precompression is based on research, in which largely rectangular panels, without openings, were tested. When irregular shapes of panel, or those with substantial openings, are to be designed it will often be possible to divide them into sub-panels, which can then be calculated using the rules given in clause **36** (see Figure 26). Alternatively, an analysis, using a recognized method of obtaining bending moments in flat plates, e.g. finite element or yield line, may be used and these can then be used instead of the moments obtained from the coefficients given in Table 9.

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 code.

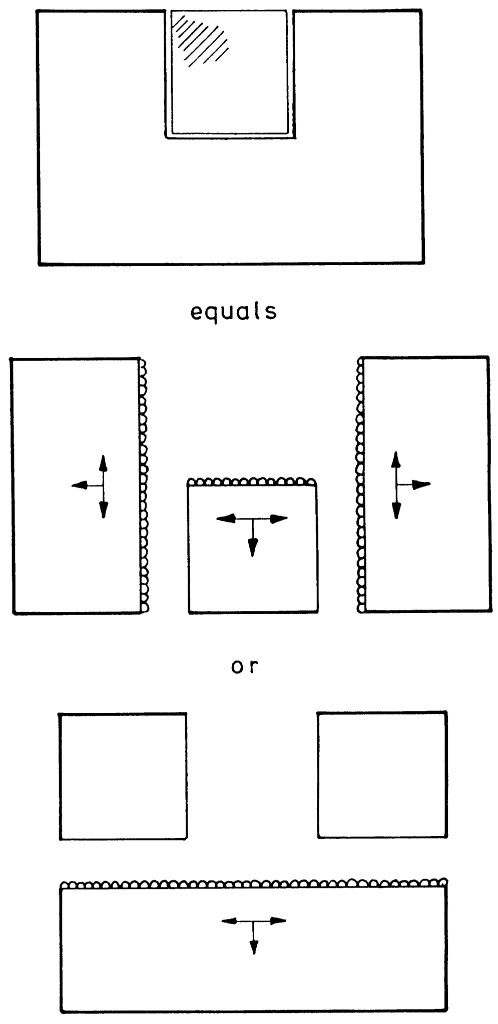


Figure 26 — Dividing a panel into parts to allow for openings

Publication(s) referred to

BS 12, *Specification for Portland cement.*

BS 146, *Specification for Portland blastfurnace cements with strength properties outside the scope of BS EN 197-1.*

BS 146-2, *Metric units.*

BS 187, *Specification for calcium silicate (sandlime and flintlime) bricks.*

BS 743, *Specification for materials for damp proof courses.*

BS 1014, *Specification for pigments for Portland cement and Portland cement products.*

BS 1217, *Specification for cast stone.*

BS 1243, *Specification for metal ties for cavity wall construction.*

BS 3921, *Specification for clay bricks.*

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

BS 4027-2, *Metric units.*

BS 4551, *Methods of testing mortars, screeds and plasters.*

BS 4721, *Specification for ready-mixed building mortars.*

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

BS 4887, *Mortar admixtures.*

BS 5224, *Specification for masonry cement.*

BS 5502, *Buildings and structures for agriculture.*

BS 5502-22, *Code of practice for design, construction and loading.*

BS 5628, *Code of practice for use of masonry.*

BS 5628-2, *Structural use of reinforced and prestressed masonry*[4].

BS 5628-3, *Materials and components, design and workmanship.*

BS 6073, *Precast concrete masonry units.*

BS 6073-1, *Specification for precast concrete masonry units.*

BS 6399, *Loading for buildings.*

BS 6399-1, *Code of practice for dead and imposed loads.*

BS 6399-2, *Code of practice for wind loads.*

BS 6399-3, *Code of practice for imposed roof loads.*

BS 6457, *Specification for reconstructed stone masonry units.*

BS 6649, *Specification for clay and calcium silicate modular bricks.*

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

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

BS 8110, *The structural use of concrete.*

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

CP 111, *Structural recommendations for loadbearing walls.*

DD 34, *Clay bricks with modular dimensions.*

DD 59, *Calcium silicate bricks with modular dimensions.*

DD 86, *Damp proof courses.*

DD 86-1, *Methods of test for flexural bond strength and short term shear strength.*

DD 86-2, *Method of test for creep deformation.*

DD 140, *Wall ties.*

DD 140-1, *Methods of test for mortar joint and timber frame connections.*

DD 140-2, *Recommendations for design of wall ties.*

[4] Referred to in the foreword only.

Index

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

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