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HANDBOOK TO BS 5628: STRUCTURAL USE OF MASONRY

Part 1: UNREINFORCED MASONRY

CONTENTS

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Part 1: Unreinforced Masonry

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The Brick Development Association

FOREWORD
by Professor A. W. Hendry

The purpose of BS 5628 is to provide essential data and guide-lines for the structural design of masonry buildings, based on accepted good practice and research results.

The writers of codes of practice are invariably faced with the problem of having to write succinct and rather specific statements, without being allowed to spell out the frequently imprecise basis of these statements. Similarly, it is not possible in a code to envisage all the possible situations to which a particular clause may be applied by others, nor are the drafters permitted to set out the limitations inherent in a particular statement. Finally, there is the dilemma of making rules which are sufficiently clearly defined, but which do not, at the same time, limit innovation.

To overcome these problems, it is extremely helpful for codes to be accompanied by explanatory documents, written by authors who have played a large part in the drafting process and who are, therefore, aware of the background to the code formulations. This is the more necessary when, as in the case of BS 5628, the code is more than a revision of a previous edition, long familiar to designers.

This handbook includes detailed discussion of every clause in the code, supported by references from the now very extensive literature on masonry construction and by a large number of illustrative examples. The latter are presented in full numerical detail, and cover practically all likely design problems.

The authors are exceptionally well qualified for the task which they have undertaken. Mr Haseltine is a consulting engineer of many years standing who has been responsible for the design and construction of many major buildings in masonry, who was an extremely active member of the drafting committee of BS 5628, and has also played a large part in the formulation and reporting of extensive research projects – the results of which were essential in the preparation of the code. Subsequent to the publication of the new code, Mr Haseltine took over chairmanship of the code committee.

Dr Moore, a senior member of the staff of the Building Research Establishment, also played a large part in the lengthy process of drafting BS 5628, on the basis of his extensive knowledge of the research background to the masonry code, much of which originated at BRE.

This handbook is, therefore, authoritative on the basis of the authors' direct knowledge of the subject, and of the discussions in the drafting committee. It is also practical because of their extensive experience in masonry design, construction and research. Users of BS 5628 will be much indebted to the authors of the handbook for their work in clarifying the basis of the code, and its application to practical design.

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INTRODUCTION

1

A draft Code of Practice for 'Structural Recommendations for Load-bearing Walls' was circulated for comment in 1946. It had been prepared by the Institution of Structural Engineers on behalf of the British Standards Institution (BSI). It included a general section on loadbearing walls, followed by sections on masonry, including brickwork (unreinforced), on reinforced masonry and on concrete cast in situ. After appropriate discussion and revision the document was published in 1948 as CP 111.

The first revision of this Code of Practice was published in 1964. The main change affecting brickwork was an increase, usually substantial, in the permissible stresses. The basic stresses were altered slightly but the reduction factors for slenderness were made less onerous and were extended to include the effects of eccentric loads.

From the beginning, the code was based on the assumption that normal principles of structural design would be used to assess the loads produced by a structure on its masonry elements. The detailed clauses of the code then gave guidance to enable wall thicknesses to be determined in relation to stresses that were considered to be safe and permissible, based on an assessment of experimental data and practical experience. The experimental tests were carried through to collapse in most cases, so that even at that time design was related effectively to ultimate stresses. The changes in 1964 reflected an increased body of experimental knowledge coupled with satisfactory results from using the first code.

A further revision was published in 1970 as part of the programmed change in the construction industry from Imperial to SI units. The existing code was renamed CP 111:Part 1:1964, still in Imperial units, and CP 111:Part 2:1970 was published as its SI equivalent. Rounding of values occurred on conversion but, although there were other minor changes, the new code did not constitute a technical revision. However a proposed revision of the code had been published as a 'draft for comment' but in July 1970 it was decided to delay publication so that the code could be redrafted in limit state terms which had been developed over the previous 10–15 years. A 'draft for comment' of the proposed unified concrete code had been published already in 1969. Following assimilation of the comments CP 110 was published in 1972 and effectively superseded those clauses of CP 111 dealing with plain concrete walls.

A number of the changes envisaged in the new draft of CP 111 were still appropriate, quite separately from the change to the limit state approach, and amendments were issued to both parts of CP 111 in 1971. The changes included the amendment from nominal to actual thickness, further changes to the reduction factor for slenderness, an increase in the maximum slenderness ratio, and a change in the permissible shear stress. Minor editorial amendments were published in 1976 and at the same time CP 111:Part 1:1964 was withdrawn and Part 2 was renamed CP 111:1970.

After the 1970 draft revision had been converted to the limit state philosophy, together with the inclusion of some new technical matter, a further 'draft for public comment' was issued in 1974 and was discussed extensively at an Institution of Structural Engineers symposium. A large volume of comment was received and sub-committees sat for 2 years revising the text. It was, perhaps, inevitable that the new accidental damage and lateral loading sections took the longest to finalise, especially as research into lateral loading was proceeding during the drafting; indeed this research is still continuing, and refinement of the design methods may be possible in the future.

With the requirement that British Standards are approved by committees unanimously, considerable discussion was needed on the combined work of the four sub-committees before final agreement to publish was reached in May 1978; the code was published in October 1978. In due course, it may be expected to be deemed to satisfy the Building Regulations. Two amendments, largely containing corrections, have now been published.

Work has started on Part 2 to the code which will deal with reinforced and, possibly, prestressed masonry. After Part 2 has been published it will be possible to withdraw CP 111, but in the meantime both codes will exist side by side.

This handbook sets out (in Chapter 2) to give background and explanatory material on many of the clauses in the new code; where possible, references have been given so that the source of information can be examined by those who wish so to do.

In Chapter 3, those formulae needed for design are given together with detailed recommendations for the analytical aspects of design. Chapter 4 consists of worked examples, extensively cross-referenced to the code clauses and to the explanatory sections of this handbook.

Acknowledgements

The grateful thanks of the authors are extended to the British Standards Institution for permission to publish extracts from BS 5628: Part 1 and to Neil Tutt of Jenkins and Potter for all his work on the calculations.

BACKGROUND AND EXPLANATION

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The numbers of the paragraphs which follow correspond to the clause numbers in the code. References to the code in the text are prefixed with the word 'code'.

SECTION 1: GENERAL

Basic guidance for the application of the code of practice is given under five headings which indicate the context in which the code should be used and set down certain criteria relating to its objectives.

1. Scope

In common with most recent structural codes of practice, a presumption is made about the qualifications of the designer and the supervisor during construction. This condition arises, not because the code is necessarily more involved or sophisticated than hitherto, but because by its nature a code does not purport to be a comprehensive design manual. It gives detailed guidance in some places, but in others only draws attention to factors to which the designer should attend when devising a structural scheme for a specific building. In doing so the designer will have of necessity to make assumptions appropriate to the circumstances in addition to those inherent in the detailed recommendations of the code. In order to ensure the satisfactory realisation of a design it is essential that these assumptions are justified in practice by the provision of the necessary supervision.

2. References

The following standards publications are referred to in the code.

BS 12	Specification for ordinary and rapid-hardening Portland cement
BS 146	Portland-blastfurnace cement Part 2 Metric units
BS 187	Specification for calcium silicate (sandlime and flintlime) bricks
BS 743	Materials for damp proof courses Metric units
BS 1014	Pigments for Portland cement and Portland cement products
BS 1180	Concrete bricks and fixing bricks
BS 1217	Cast stone
BS 1243	Metal ties for cavity wall construction
BS 2028, 1364	Precast concrete blocks
BS 3921	Clay bricks and blocks
BS 4027	Sulphate-resisting Portland cement Part 2 Metric units
BS 4551	Methods of testing mortars and specification for mortar testing sand
BS 4721	Ready-mixed lime: sand for mortar
BS 4887	Mortar plasticizers
BS 5224	Specification for masonry cement
BS 5390	Code of practice for stone masonry
CP 3	Code of basic data for the design of buildings Chapter V. Loading Part 1 Dead and imposed loads Part 2 Wind loads
CP 110	The structural use of concrete
CP 111	Structural recommendations for loadbearing walls
CP 121	Walling Part 1 Brick and block masonry
CP 2004	Foundations
DD 34	Clay bricks with modular dimensions
DD 59	Calcium silicate bricks with modular dimensions

3. Definitions

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.

The use of 'actual dimensions' follows its introduction into CP 111 by the 1970 amendment. The expression 'solid walls' includes all those types defined in clause 3.21, except for 'cavity wall'.

3.2 category 1 building

A building having 4 storeys (including basement storeys), or less.

A classification introduced in relation to design to resist accidental damage (section 5).

3.3 category 2 building

A building having 5 storeys (including basement storeys), or more.

Complementary classification to that in clause 3.2.

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.

This definition follows from the recognition that real values of loads will have a statistical distribution about a mean value. The 5% fractile is an arbitrary cut-off, used by most if not all limit state codes, as a means of characterising the tails or extreme values of the distribution – hence 'characteristic' value. Probabilistic concepts are thereby introduced into the design. Depending on the effect of a load on stability it is necessary to distinguish maximum or minimum extreme values. The practical method of obtaining characteristic loads at present arises from the lack of adequate statistical data about some types of loading.

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%.
The meaning of 'characteristic' is the same as discussed under clause 3.4 except that, for strength, only minimum values are of interest to a limit state design. In this context, however, the reference to 'test results' implies that the characteristic strength has been determined from tests on specimens of masonry constructed in the laboratory.

3.6 compressive strength of structural units
For the normal category of manufacturing control, the compressive strength of structural units is 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.

This general statement acknowledges the range of methods specified for determining the compressive strength of different types of structural unit. The significance of the two categories of manufacturing control of quality is discussed below.

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

In the case of a member with a cavity, this definition should be applied to the overall thickness of the member unless only one leaf is loaded, when it should be applied to the loaded leaf.

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.

This definition draws attention to a type of element which was not covered by CP 111. However, its definition, in common with that of 'load-bearing walls' in clause 3.14, is not mutually exclusive. Between the extremes of high lateral load and only self-weight vertically, and high vertical load only, all combinations of lateral and vertical loads may be envisaged. The appropriate definition and design approach must be decided depending on the load which predominates.

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.

This definition emphasises the requirement for lateral supports to resist *movement*, although in the relevant design clauses (28.2) the forces necessary to prevent movement are usually considered. The elements providing lateral (ie horizontal) support may be arranged in either vertical planes eg piers and buttressing walls, or horizontal planes eg roofs and floors.

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.

The reference to reinforced masonry recognises the scope of masonry beyond Part 1 of BS 5628.

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.

Although a common term in structural engineering it has not been applied previously to masonry. It now refers to the ratio of the flexural strengths of masonry determined in two directions at right angles to each other. These directions are depicted in Figure 1 and are called 'parallel' when the failure surface due to bending is parallel to the bedding joints, and 'perpendicular' when failure occurs at right angles

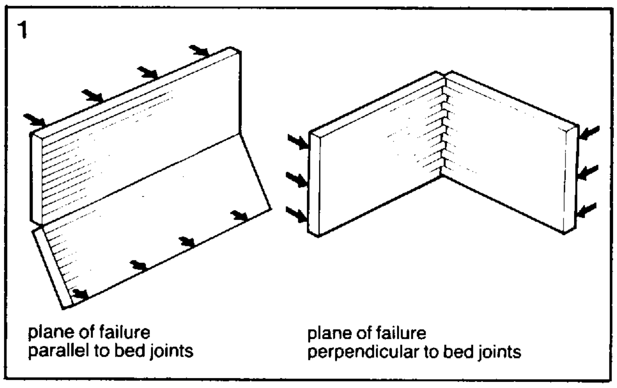


Figure 1 Orthogonal failure directions.

to the bedding joints. This terminology should not be confused with that used to describe the moments which cause failure and which act in planes at right angles to the failure surfaces, and which are therefore opposite to the definition used in this code.

The strength in the parallel direction is usually the lower so that the orthogonal ratio of the masonry itself is usually less than unity.

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, *page* 50

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.

This definition implies that structural units are right parallelepipeds. However, clause 7 'Structural Units' includes BS 5390: Stone masonry which covers random rubble masonry ie non rectangular units, to which reference is also made in clause 23.1.9 for characteristic compressive strength.

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.

The name given to this form of wall may vary locally but the construction is designed essentially to achieve a finished wall fair-faced on both sides.

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.

The definition of single-leaf walls includes walls wider than the minimum plan dimension of the structural units used. The definition of cavity walls therefore includes leaves of different thicknesses. The non-loadbearing material in the cavity will usually be a thermal insulating material which may not necessarily occupy the full width of the cavity.

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.

Although defined here and referred to in clause 29.6 the more usual application for a grouted cavity wall is in reinforced masonry.

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 walls
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.

Although panels ranging from a few stack-bonded units to a half-storey height wall could be included in this definition the main size of wallette envisaged is similar to that described in Appendix A.3 for the determination of flexural strength, ie several courses high and a few units long.

4. Symbols

For ease of reference the symbols used in the code are reproduced here. The notation follows closely that used in CP 110 based on the following convention:
Roman capitals to denote forces, area or related functions, and constants.
Roman lower case to denote length and force per unit area or per unit length.
The meaning of subscripts is more variable but k denotes a characteristic value and m denotes a mean value except when subscript to a Greek symbol.
Greek lower case to denote factors or coefficients.
The use of L for length is an exception and a particular difference from CP 110 is the use of t for width or thickness and h for height.

A	horizontal cross-sectional area
B	width of a bearing under a concentrated load
b	width of column
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
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
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
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

The concept of 'good practice' embodied in BS 5628: Part 1 does not necessarily represent an exclusive approach to the design of masonry structures and to the use of appropriate materials. Such a rigid view would prejudice and inhibit development and innovation, or even individual preference. However, the code does set or indicate required standards and guiding principles which may be used as bases of comparison against which to judge the use of alternative procedures and materials.

A parallel may be drawn with Codes or Schedules which may be 'deemed to satisfy' the requirements of the Building Regulations but which do not provide necessarily exclusive solutions to particular requirements.

The prime constraint on the use of alternative methods or materials is that their suitability should be judged on the basis of tests which are designed to represent as far as possible the significant factors which would influence their performance in a real building.

The code contains specific procedures in Appendix A for determining characteristic compressive and flexural strength as well as for testing mortars.

SECTION 2. MATERIALS, COMPONENTS AND WORKMANSHIP

As a logical sequel to clause 5 this section defines the standards of materials necessary to achieve masonry construction of the required quality and reliability envisaged by the code. It does so by referring to the principal existing British Standards.

6. General

Although this code relies on the recommendations in CP 121 and BS 5390 to provide generally acceptable standards, in many cases, to achieve higher quality brickwork, certain clauses in CP 121 (see particularly clause 27, below) will require particular emphasis or amplification by the designer.

7. Structural units

The value of a particular property of a structural unit, especially compressive strength is strongly dependent on the method of testing. Therefore use of the recommendations in the code can be justified only if unit strength or other property has been determined in accordance with the appropriate British Standard.

8. Laying of structural units

8.1 General

Speculation may be raised over the precise definition of bed faces, stretcher and end faces in the 'General' clause 8.1, since it could be argued that the horizontal surface on which a unit is laid constitutes its bed face, irrespective of what it might be called when laid in another attitude. However, the express purpose of the clause is to draw attention to the different attitudes in which units may be used and to point the need for tests to be performed in that attitude which relates to the use of the unit in the masonry being designed.

When reference is made in clauses 8.2, 8.3 and 8.4 to laying on a full bed of mortar attention is drawn (by a footnote) to the need to allow for the effects of raking out a joint. Whether this is done prior to pointing or as a finished feature, the thickness of the wall used for calculating stresses and loads should take into account this reduction. Although the warning as stated refers only to frogged and perforated bricks it should be taken as applying to all bricks and blocks.

Reference to cellular bricks is not included. Normally they will be laid on a full bed of mortar with their aperture lowermost and if so used should be tested of course in that attitude.

8.2 Bricks with frogs

While it would not normally be practicable or desirable to fill the cells of a cellular brick with mortar, the same is not necessarily true of frogged bricks. The most efficient use of such units in developing the maximum strength possible in a wall requires full and proper bedding on mortar and the filling of any frogs in the bricks. In practice, it may not be possible to ensure that the lower frog of a double-frogged brick is filled, and less onerous strength requirements may make it acceptable to place single-frogged bricks frog down and with the frog therefore only partially filled. Under these circumstances, strength tests according to BS 187 or 3921 should be performed on bricks with unfilled lower frogs.

8.3 Perforated bricks

8.4 Hollow and cellular blocks

The titles of these clauses emphasise the fact that the terms brick and block imply relative shapes and sizes of units without distinction of material. It just happens that under current conditions in Britain most bricks are clay or calcium silicate based and most blocks are cement based.

Simms L G,
Frog-up or frog-down brickwork compared.
The Builder, 191(5917) p. 329–31, 1956.

Hodgkinson H R,
The compressive strength of brickwork tested in orthogonal directions.
B Ceram R A, Report to BDA.

9. Rate of laying

Although the weight of an excessive height of masonry may squeeze out fresh mortar there are other more serious effects which require a limitation to the rate of laying. A higher wall is more vulnerable to minor disturbance during setting which can lead to a rounded profile to the hardened mortar and reduced bond at the edges of the units. The importance of these adverse effects will be magnified when eccentric vertical loads are supported or when the flexural strength of masonry is to be relied upon. Installing cavity wall ties as both leaves are taken up together also has a stabilising advantage for the two leaves.

Haller P,
Die technischen Eigenschaften von Backstein-Mauerwerk für Hochhauser.
Schweizerische Bauzeitung, 76(2) p. 411–419, 195.
Also available as BRS Library Communication 70:
The properties of loadbearing brickwork in perforated fired bricks for multi-storey buildings.

Haller P,
Load capacity of brick masonry.
Designing, engineering and constructing with masonry products, Ed. F B Johnson Gulf Publishing Co, Houston, Texas, 1969.

10. Forming of chases and holes

11. Pallet slips

These apparently minor modifications may be carried out relatively easily on site and without reference to any supervisor. In many cases they will be admissible but in extreme cases may have a serious weakening effect on the strength of the masonry. The designer should always be consulted before such modifications are made.

Fisher K,
The Effect of Chasing on the Compressive Strength of Brickwork.
Proc 3rd Int Brick Masonry Conf. Ed. L Foertig and K Göbel Bonn Bundesverband der Deutschen Ziegelindustrie, 1975.

12. Damp-proof courses

Despite the widespread use of damp-proof courses in masonry elements, their structural properties have not been studied widely, nor do the BS specifications provide a structural performance requirement. There is some limited data available to supplement that which may be obtained from the manufacturers. Reference may also be made to BDA and BCRA who are obtaining experimental data. The principal factors to be considered are (a) resistance to squeezing out due to compressive loads, (b) ability to resist shear stresses, and (c) adhesion to mortar so that flexural stresses may be transmitted.

Plowman J M & Smith W F,
The selection of damp-proof course material for loadbearing structures.
Proc 2nd Int Brick Masonry Conf. Stoke-on-Trent 1970. Ed. West H W H and Speed K H BCeramRA 1971.

Damp-proof courses.
Building Research Station Digest 77. HMSO, 1971.

13. Wall ties

BS 1243 provides a specification for the geometry of certain configurations of ties and minimum strength requirements for a range of basic materials together with recommendations for achieving some resistance to corrosion. It does not provide a structural or durability performance specification as such.

Choice between the various types of tie therefore requires first an assessment of their structural performance and that demanded by the particular design circumstances. A major difficulty is that maximum load-sharing between leaves of a cavity wall requires high stiffness on the longitudinal axis of the ties, eg a vertical twist tie, but differential in-plane movements between leaves, particularly when they are of different materials, require low flexural stiffness, eg a double triangle tie. Clause 36.4.5 considers the requirement further.

It is clear that the necessary circumstances for corrosion of zinc, namely oxygen and water, may be present in the outer leaf of a cavity wall for significant periods. Any protection afforded by alkalinity of the mortar will be lost due to carbonation in a few years. Therefore, serious consideration should be given to the selection of ties of adequate durability, particularly when a life of at least 60 years is required during which the minimum margin of safety is not reduced at all.

Galvanised ties to BS 1243 may not be acceptable, particularly those fabricated from pre-galvanised wire. It is worth noting that they are not permitted by the London Building (Constructional) By-laws for use in buildings greater than three storeys in height.

De Vekey R C,
Corrosion of steel wall ties: recognition, assessment and appropriate action.
BRE IP28/79.

14. Cements

The references to cements have been limited to Portland cement (ordinary and rapid-hardening), Portland blast-furnace cement, sulphate-resisting Portland cement and masonry cement, which are the most relevant to mortars. Although BS 4248 for Supersulphated cement is included in CP 121 it is used scarcely if at all now for mortar, cement to BS 4027 usually having adequate resistance to sulphates. The low heat properties of BS 4248, along with low-heat cements to BS 1370 and 4246, are not necessary for mortars.

15. Mortars

15.1 General

Identification of a particular mix proportion for a type of mortar is given now by the term mortar 'designation'. This term, already used in CP 121, is an attempt to encourage the designer to assess the desired characteristics for a mortar for a particular application, and not to assume that a higher cement content necessarily means a better mortar. Indeed, it is generally advisable to consider using as weak a mortar as possible consistent with adequate strength and durability. However, extreme care is needed to ensure that the proportion of sand is not exceeded, and that the correct amount of cement is present.

Since a wide range of particle sizes may be present in building sands which conform to BS 1200, the sand required by CP 121 for mortars, the compressive strength values for mortars in code Table 1 are essentially lower limits. It is possible that some sands conforming to BS 1200, and used in the proportions specified in code Table 1, may not yield mortars of the required strength, although this shortcoming will be apparent only when site control of mortar is being used or when determining masonry strength by test (see code, Appendices A.1 & A.2 respectively).

A survey by CIRIA has shown that, in many areas, sands conforming to BS 1200 are not available, but that other sands have produced apparently satisfactory results. The practice of using sands which do not conform to the BS seems acceptable provided that the strength requirements can be demonstrated to be met, for example, for a range of Scottish sands.

It must be appreciated that the compressive strength data in code Table 2 are based on tests carried out on specimens built in mortars, primarily cement:lime:sand, conforming to the designations in code Table 1.

15.2 Ready mixed mortars

Although there is not thought to be anything inherently wrong with the use of retarded ready-mixed mortars care must be taken to ensure that they are used only during the allowable period of workability to avoid premature setting. Due to the extended period before final set the vulnerability to disturbance is increased and there is a greater risk of deformed bed joints and reduced bond, as discussed under clause 9.

Ragsdale L A & Birt J C,
Building sands: availability, usage and compliance with specification requirements.
CIRIA Report 59, 1976.

Sinha B P,
Strength of mortar for brickwork.
Proc Inst Civ Engrs Part 1 v60 pp 655–662 1976.

Skeen J W,
The strength of brickwork built with plasticised (aerated) mortars.
Trans Brit Ceram Soc 62(8) 631 1963.

Sneck T,
Winter masonry construction.
Proc 1st Canadian Masonry Conf, Calgary 1976.

16. Colouring agents

The recommendations of this clause should be followed closely to avoid significant reduction in strength of the mortar, particularly when using cement:lime:sand mortars.

Thomas K Coutie M G and Pateman J,
The effect of pigment on some properties of mortar for brickwork.
Proc 2nd Int Brick Masonry Conf. Stoke-on-Trent 1970. Ed West H W H and Speed KH B Ceram RA 1971.

17. Plasticisers

The warnings acknowledge the wide range of compounds available as mortar admixtures. Usually only small quantities are required so that the resulting change in mortar properties is often very sensitive to the concentration. Control is particularly important since excess plasticiser may be to the bricklayer's immediate advantage at the expense of an excessively porous mortar of reduced durability, reduced strength and reduced bond to the masonry units.

Concrete Admixtures: Use and Applications:
Chapter 6 'Mortar Plasticisers'. Ed Rixom M R.
The Construction Press London 1977.

18. Frost inhibitors

The extremely harmful effect of chlorides in accelerating the corrosion of metals cannot be emphasised too strongly. In any case, calcium chloride is not an effective frost inhibitor. Its action is to accelerate setting, during which the temperature rises so that the risk of freezing is reduced. In practice, however, the masonry units with which the mortar is in contact will be usually at a low temperature as well. Their thermal mass is so relatively large that it will swamp any heating effects in the relatively thin mortar joints.

Wrenn H & Butterworth B,
A note on the effect of chlorides on the sulphate content of bricks.
Trans Brit Ceram Soc vol xiv p 412 1949.

Newman A J,
Concreting and bricklaying in cold weather.
National Building Studies Bulletin 3 HMSO 1964.

Publications of Engineering Division (Structural) BRS.
BRS Library Bibliography 195, 1966.

Thomas K,
Bricklaying under winter conditions.
BDA Special Publication 1971.

SECTION 3. DESIGN: OBJECTIVES AND GENERAL RECOMMENDATIONS

19. Basis of design

The designer familiar with CP 110 will be accustomed to the concept of the limit state approach which is applied now in this code to the design of masonry members. Many structures contain concrete elements as well, so a welcome uniformity of approach is afforded.

The basis of limit state design is that the designer should consider all the likely ways in which a structure or element could fail to perform its required function and should then ensure that there is an acceptable probability that failure will not occur. The approach at once acknowledges the inherent variability of building materials, construction processes and applied loads, and gives the designer the opportunity to take advantage of methods or information which improve his control or knowledge of the variability.

The modes of failure are the so-called *limit states* and should be considered in two groups:
(1) *ultimate* limit states: compressive failure, tensile failure, shear failure, flexural failure and
(2) *serviceability* limit states: cracking, deflection, displacement.

In the case of masonry, permissible stresses have been related nearly always to ultimate stresses so that the limit state approach may be regarded as a more refined method with greater objectivity. Although the serviceability limit state is often of importance in masonry structures its control usually lies with detailing covered by CP 121, and general layout, rather than by structural design. However, when masonry resists lateral loads by arching (clause 36.4.4) cracking usually occurs at a load very much lower than the ultimate load.

In practice design is performed on individual elements of a structure, taking account of interactions between adjacent parts. However, this procedure does not ensure necessarily that the overall structure cannot reach a limit state, since at the larger scale there may exist mechanisms of behaviour not applicable to the elemental scale. Attention is drawn in clause 20 to these wider considerations.

One of the key operations in applying the limit state approach lies in assessing probabilities of reaching a particular limit state, of judging their acceptability, and of incorporating them explicitly into a design procedure. Methods of varying sophistication are available. A three level classification has been adopted by the International Association for Bridge & Structural Engineering (IABSE) according to the type of approximation made and the way in which reliability is defined.

In the absence of a full or adequate statistical knowledge of the parameters involved in structural design, a simplified (Level 1) approach has been used in BS 5628 based on a *partial safety factor format*. This format uses *characteristic values* of the parameters which are based ideally on statistical data and represent a certain probability of the particular value being achieved (see for example definitions 3.4 and 3.5, and below). These characteristic values are then modified by *partial safety factors* which provide for a measure of further uncertainties existing in a real structure. Theoretically, knowing the full statistical distribution of a parameter, it is possible to calculate a partial factor corresponding to a given probability of occurrence eg 10^{-3} or 10^{-4}, of an extreme value. The partial factors so derived may be presented as a series of curves (Figure 2) as functions of the coefficient of variation of the selected parameter. Clearly an appropriate factor could be chosen for every parameter so that in the design relationship the combination of probabilities represented by the various factors would yield the required probability of failure for the whole structure. The simplest possible example of this approach is shown in Figure 3 for the interaction of two parameters eg load, F and strength, S. In practice even this apparently simple combination of probabilities to give an overall reliability is not straightforward (Beech). Taking into account the lack of detailed knowledge of the statistical variation of all the parameters it is necessary to resort to a more subjective assessment of values for partial factors (see clauses 22 and 27).

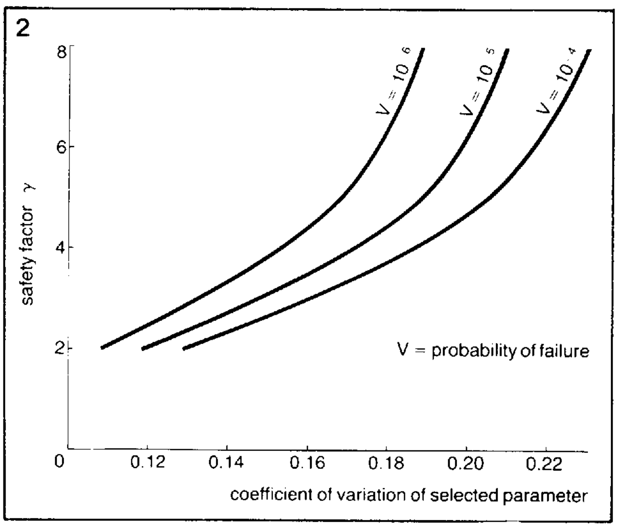
The factored characteristic values are called *design values*. The objects of design are then achieved usually by ensuring that the design strength of a member is not less than its design load. In the simplest terms this relationship can be represented for each element by the inequality:

$$\gamma_f F_k \leqslant \frac{S_k}{\gamma_m}$$

in which γ_f represents a partial safety factor expressing uncertainty in the value of the characteristic load F_k and γ_m represents a partial safety factor expressing uncertainty in the value of the characteristic strength S_k.

This expression may be arranged readily as $F_k \leqslant \frac{S_k}{P}$ where $P = \gamma_f \times \gamma_m$ which is equivalent to the single

Figure 2 Relationship between safety factor and coefficient of variation of one variable for various probabilities of failure.



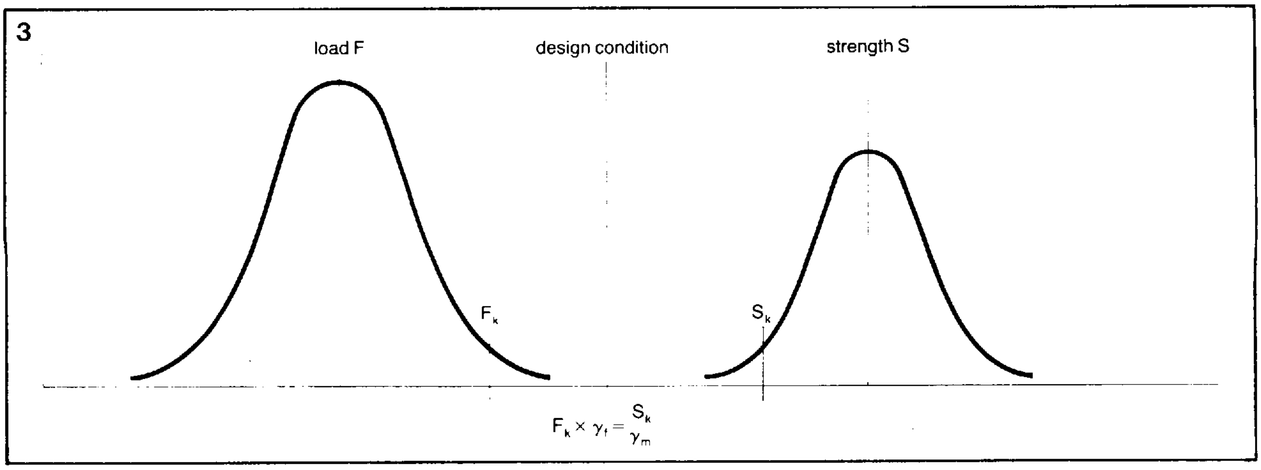


Figure 3 Simple design relationship between two probabilistic variables.

factor of safety incorporated in the permissible stress approach of CP 111. But whereas this factor existed only implicitly in the permissible stresses which were quoted, the opportunity now exists to accommodate explicitly a range of factors related to current knowledge about loads and materials and to accommodate other areas of ignorance.

There are some cases in which design involves achieving an equilibrium of loads without involving strength of materials as a prime factor. Examples are provided by free-standing walls with no allowable flexural strength for which failure occurs by toppling or by the lateral strength of walls subject to significant axial loading. In these cases appropriate partial factors must be assessed for the restoring forces in addition to the disturbing loads.

As already defined in clauses 3.4 and 3.5 characteristic values are a means of defining extreme values likely to occur in populations of measurements. By common convention a 5% limit has been chosen and indicates that 1 in 20 of the values may lie above or below the characteristic value, depending on whether an upper or lower limit has been selected (Figure 4).

The characteristic value may be estimated from the mean value by various methods depending on the number of measurements, their standard deviation (or as expressed by coefficient of variation) and the form of the statistical distribution of individual measurements about the mean. Detailed discussion for the derivation and accuracy of the following relationships may be found in statistical treatises. The relationships relevant to this code are stated here for convenience.

(a) 2 test results: in clause A.2.7 the characteristic compressive strength f_k may be calculated from the results of two wall tests to failure by the following:

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

where F_m is the mean of the maximum loads carried by the two test panels, the other variables being functions of the test walls. If the relationship in (b) below were applicable the divisor 1.2 would imply a coefficient of variation for the test results of about 10%.

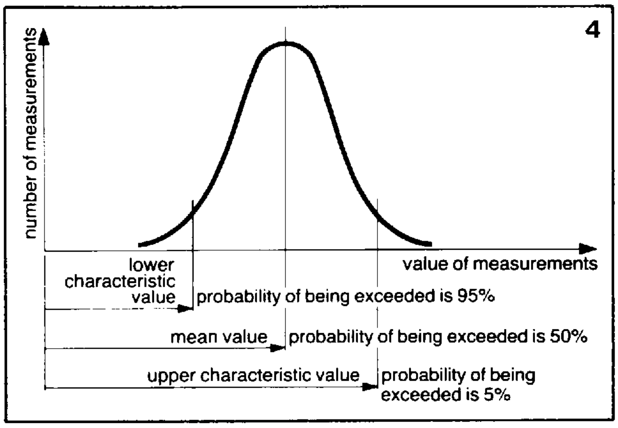


Figure 4 Characteristic values.

(b) 30 test results: most discussions of characteristic value assume that the population measured may be described by the normal or Gaussian distribution, which is symmetrical about the mean. In many cases this assumption is reasonable and $f_k = f_m \pm 1.64s$ where f_m is the mean value of the sample, s is the standard deviation of the sample, and the constant 1.64 relates to the 5% level of probability. For the same distribution different constants would relate to different levels of probability. Characteristic compressive strength values given in code Table 2 have been derived from mean values on the basis of this expression.

(c) 10 test results of high variability: when the coefficient of variation is large, say 15% or more, a normal distribution becomes an increasingly poor fit to the spread of results, not least because it implies negative values which is nonsense for the physical quantities under consideration here. A skew, positive only, distribution is required and one such is the log-normal distribution. In the case of flexural strength tests, it has been shown by Beech to be a reasonable representation and is scarcely more difficult to handle. The method of calculation is given in code Appendix A.3.3 and is similar to the usual calculation for standard deviation except that experimental values are replaced by their natural logarithms. The constant for obtaining characteristic value from mean value in terms of logarithms becomes 1.922.

Turning in more detail again to the partial safety factors it is as well to be clear exactly what types of variability are included in the load (γ_f) and material (γ_m) factors.

γ_f takes account of:

(a) possible unusual increases in load beyond those considered in deriving the characteristic load; for example, use of statistically inadequate data.

(b) inaccurate assessment of effects of loading, and unforeseen stress redistribution within the structure; the inclusion of this element may be contested on the grounds that the effects mentioned are also a function of the material forming the structure rather than the loads on it. However, the balance of current opinion favours allowance for these effects in γ_f.

(c) the variations in dimensional accuracy achieved in construction; although statistical data on the accuracy which is achieved in practice is now available for some forms of construction in BS 5606 its effect on structural reliability is not known so that it cannot be quantified separately.

Some authorities also include 'the importance' of the limit state being considered, that is in relation to the social or economic consequences associated with it. In the absence of any quantifiable data this aspect is normally ignored.

The factor γ_m takes account of:

(a) possible difference between the strength of masonry constructed under site conditions and that determined from specimens built and tested in the laboratory.

(b) any other variation in the quality of the structure

One of the important attributes of the partial factor format in limit state design is that as new data become available to improve knowledge of some aspect of materials, loads or behaviour, the appropriate partial factor may be reduced while leaving unchanged those factors describing other aspects. In practice this flexibility does not exist yet to the extent desirable since, as enumerated above, both γ_f and γ_m include a mixture of possibly measurable factors and inherently unknowable elements.

As indicated already, no specific procedures are given for examining serviceability limit states, such as a recommendation for alternative values for the partial factors. Design consideration of these matters is less explicitly developed for masonry than, for example, for reinforced concrete. Cracking is the condition most usually to be avoided but under many forms of loading the onset of cracking is followed very soon by ultimate failure. The main exceptions are provided by combined lateral load and high in-plane loads, and stresses induced by movements as opposed to applied loads. Such movements may be caused by thermal and moisture changes in the masonry elements themselves, by differential movement between connected elements e.g. reinforced concrete frame and masonry panels, leaves of a cavity wall of markedly differing properties, or by foundation movements. Although some guidance for avoiding such problems may be found in CP 121 and various guidance notes, the key to satisfactory design for serviceability limit states lies in a correct appreciation of the possible behaviour of materials and the provision of appropriate details to accommodate movement without developing loads for which the elements have not been designed.

Aims of structural design,
IStruct London, 1969.

Cracking in buildings.
BRS Digest 75, HMSO 1970.

Macchi G,
Safety considerations for a limit-states design of brick masonry.
Proc 2nd Int Brick Masonry Conf. Ed West H W H and Speed K H Stoke-on-Trent BCeramRA 1971.

CP 110: 1972. The Structural Use of Concrete.
British Standards Institution London.

Hendry A W,
The lateral strength of unreinforced brickwork.
The Structural Engineer v 51 n2 1973.

Cranston W B,
Limit state design and its application to masonry structures.
Symposium on the Structural Use of Masonry IStructE London 1974.

Burland J B & Wroth C P,
Allowable and differential settlements of structures including damage and soil-structure interaction.
Review paper Conf on Settlement of Structures Cambridge 1974 Pentech Press London.

Beech D G,
Some problems in the statistical calculation of safety factors.
Proc 4th Int Brick Masonry Conf. Bruges 1976.

Beech D G,
The concept of characteristic strength.
6th Int Symp on Loadbearing brickwork, London 1977. Proc Brit Ceram Soc n27 1978.

BS 5606: 1978. Accuracy in building.
British Standards Institution, London.

20. Stability

Although the detailed procedures of limit state design have been introduced to masonry they still apply for the most part to individual elements with which most of the code is concerned. This clause draws attention to the vital need to consider the overall behaviour of the whole structure. It is a vital clause because some of the later detailed design recommendations rely on basic assumptions about overall behaviour and also because there have been failures of buildings due to lack of attention to and appreciation of total structural behaviour.

20.1 General considerations

For a variety of reasons the building industry has turned to the large scale manufacture of elements, among the most noteworthy being timber trussed rafters and precast concrete floor units. With their introduction, the erroneous presumption has developed that the design of complete roofs and floors need not be considered further, and that no attention need be given to the part that roofs and floors play in forming and stabilising the overall structure.

However, even though the component manufacturer may make recommendations about the way in which his product should be included in a complete roof or floor, it is the overall designer who is responsible for their correct inclusion and for consideration of their effect on the whole structure. These aspects will generally involve the provision of bracing, connections or additional reinforcement, on some aspects of which guidance is given below.

Masonry is a traditional material which lends itself to layouts on plan which may have irregular outlines and a variety of internal walls. The traditional layout has become known as cellular planform and, due to the high degree of buttressing afforded by intersecting walls, seems a desirable form of construction.

Changes in practice due to economic pressures, shortages of craftsmen and materials, changing standards for lighting, heating and appearance have lead to simpler planforms with fewer and lighter weight intersecting walls, and larger openings. The degree of redundancy afforded by cellular planforms has been eroded considerably with consequent uncertainty about the real effects on margins of safety. Research at the Building Research Establishment (BRE) is examining the interaction between components to explore in more detail their behaviour and to indicate the magnitude of any reserves of strength over and above those required to resist normal design loads.

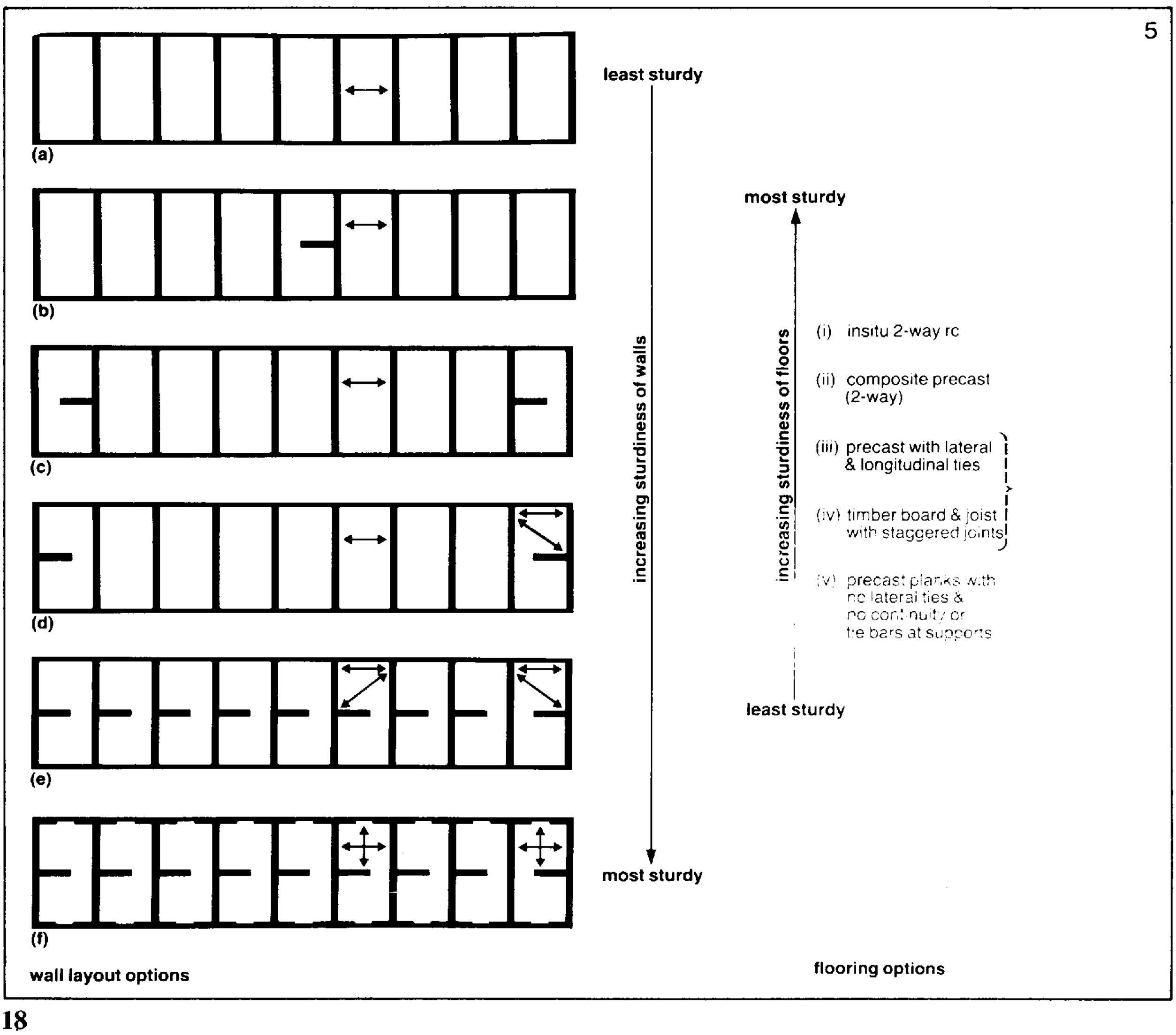
Such reserves of strength are a qualitative indication of that attribute of a building which is implied by the terms robustness or sturdiness. These terms refer to the sensitivity of a structure, or parts of it, to abnormal effects not considered explicitly in design for normal loads. Such effects include faults of design and construction eg omission of connections or unexpected foundation behaviour, and require an understanding of the use to which a building will be put. The difficulty of defining robustness, particularly for masonry buildings of less than 5 storeys, is highlighted in Figure 5 (after Sutherland). The sequence of diagrams is self-explanatory and indicates how inappropriate it is to specify rigid standards for robustness or sturdiness. However, without the designer being continually reminded of these implicit requirements there is a real possibility that proper consideration may go by default. The two paragraphs in the Code devised to embody these principles bear repetition:

'*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 draft for public comment published by BSI in

Figure 5 Comparison of wall and floor options for simple crosswall construction



1974 suggested alternative design procedures for vertically loaded walls depending on whether they were braced or unbraced, in parallel to CP 110. It was concluded that for masonry walls, which are essentially stiff and brittle, it was unwise to contemplate structural forms for buildings in which lateral movement or sidesway was permitted and that only braced structures should be recommended. This condition is a fundamental assumption to the development of the capacity reduction factor in Appendix B. It means effectively that form (a) in Figure 5 in which resistance to lateral movement is provided only bv the flexural strength of the walls is an unacceptable form of construction, even though adequate resistance might be demonstrated by calculation. Such a layout is too sensitive to adverse loads or displacements, and the code recommendation is again worth repeating:
'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.'

The general discussion on overall stability concludes with two more specific recommendations. The first states a required minimum resistance to overturning forces for the whole structure, or any part of it, up to roof level. This resistance is stated in terms of a uniformly applied horizontal load equal to 1.5% of the total characteristic dead load above any level. The method of determining the load is illustrated in Figure 6 but it is required to be resisted only by

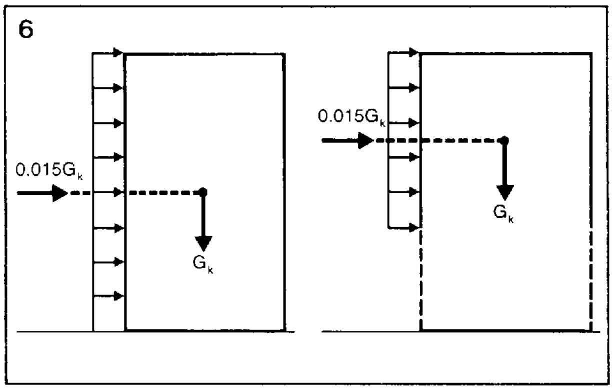


Figure 6 Application of minimum lateral load (clause 20.1a)

the structural elements. This load is presented in alternative form in clause 22 as a minimum value for wind loading which is the common horizontal load. Design to resist normal wind loads will usually satisfy this requirement for overall stability. The value of 1.5% is the same as in CP 110 and although arbitrary there seems no reason to introduce a lower figure although several other countries use only 1%.

The second recommendation focuses on connections. Since consideration of overall behaviour will involve design of connections between masonry and timber and concrete structural elements, guidance is given in Appendix C. The forms given are based on generally accepted good practice and may be used satisfactorily, often without further calculation. However, most failures are associated with shortcomings in connections and it must be considered good practice to structure a building in such a way that failure of a connection cannot lead to collapse of the whole building.

20.2 Accidental forces

The consideration of general aspects of stability and robustness has usually implied provision of some degree of redundancy to ensure that certain minimum standards are met for the structure in addition to those necessary for the design of individual elements on their own. Following the collapse at Ronan Point there was a growing belief that there was a class of extreme loading which should be considered in design and requirements were incorporated in Building Regulations for buildings of 5 storeys or more. Typical examples of the loads envisaged are gas explosions and vehicle impacts which have been shown to be the most common. Their chances of occurrence are very small and unpredictable for an individual building but foreseeable and reasonably predictable, based on historical occurrence, for buildings as a whole.

A philosophy has developed that while it is not generally economic or even possible to design structures to withstand totally the effects of likely or foreseeable extreme loads, it is possible to design structures to accommodate the effects of such loads and so limit the spread of damage. So has arisen the expression 'the extent of damage should not be disproportionate to its cause'. Here again it is difficult to set objective requirements. A given explosion which in a reinforced concrete framed structure with infill walls of varying strength might blow out only the lightest weight or weakest panel, might almost demolish a detached brick house. Yet the same brick walls at the base of a 4-storey building could well withstand the explosion.

The general exhortations given in this clause apply to all buildings although further more detailed recommendations are given in clause 37 for buildings which have to comply with the intentions of regulation D 17 of the Building Regulations. The presumption for low-rise buildings (less than 5 storeys) is that proper attention by the designer to layout and bracing to resist normal design loads will provide sufficient reserves of strength to accommodate a reasonable range of extreme loads.

20.3 During construction

The most common cause of failure during construction of masonry walls is lack of appreciation of the weakness of gable walls subject to wind loading. Temporary propping is often necessary to high gables and to ends of walls unbuttressed by returns. However, no specific documentary guidance is available.

Guidance on the design of domestic accommodation in loadbearing brickwork and blockwork to avoid progressive collapse.
IStructE London 1969.

Menzies J B & Grainger G D,
Report on the collapse of the sports hall at Rock Ferry Comprehensive School, Birkenhead.
CP 69/76 BRE 1976.

Criteria for structural adequacy for buildings.
IStructE London 1976.

Design guide for strapping and tying of loadbearing brickwork in low-rise construction.
SP 93 BCeramRA 1977.

The stability of masonry walls in housing with timber roofs.
TRADA February 1977.

Mainstone R J Nicholson H G and Alexander S J,
Structural damage in buildings caused by gaseous explosions and other accidental loadings 1971-77.
BRE 1978.

Moore J F A,
The stability of low-rise masonry construction.
Symposium on Stability of low-rise buildings of hybrid construction, London 1978. IStructE.

Sutherland R J M,
Principles for ensuring stability.
Symposium on Stability of low-rise buildings of hybrid construction, London 1978. IStructE.

21. Loads

The statistical basis of the term characteristic value is again emphasised but for most types of load it has to be accepted that there is insufficient data to enable reliable statistical derivations to be made at present. Thus the dead (G_k) and imposed (Q_k) characteristic loads are taken as the loads given in CP 3 Chapter V: Part 1.

The characteristic wind load (W_k) is obtained from CP 3: Chapter V: Part 2. In CP 3 the wind is presented in terms of a return period for the wind speed associated with various levels of probability, so that 'characteristic' is a more reasonable term. It should be noted that this load is derived from the design wind speed in CP 3. It should not be confused with the design wind load which is the characteristic wind load multiplied by γ_f (see clause 22).

In referring to CP 2004 for lateral earth pressure on foundation walls, care should be taken to make allowance for the difference in approach to safety factors between the two Codes.

22. Design loads: partial safety factor, γ_f,

The four sections (a)–(d) give the combinations of loading which should be considered by the designer, section (d) applying only to extreme or accidental loading conditions.

The loads given are design loads and comprise a number, the partial safety factor γ_f, multiplied by the characteristic load. In some cases there are alternative values for the design load or partial safety factor.

The values of γ_f for normal loads are shown in Table 1 which gives the value or values of γ_f to be associated with dead, imposed and wind load for the three combinations of these loads which should be considered. These values follow CP 110 except that in combination with imposed load CP 110 suggests a lower alternative value of 1.0 G_k. In BS 5628 this value has been reduced to 0.9 G_k on the basis that if the principle of favourable or unfavourable load combinations is accepted,

Table 1. Partial safety, γ_f for normal loads

Load combination	γ_f, Partial factor to be applied to load:		
	G_k	Q_k	W_k
Dead + imposed	0.9 or 1.4	1.6	—
Dead + wind	0.9 or 1.4	—	1.4*+
Dead + imposed + wind	1.2	1.2	1.2

**If 0.015 G_k is greater than the design wind load, 0.015 G_k should be used.*
+ If the wall may be removed without affecting the stability of the structure in any way γ_f may be taken as 1.2.

together with uncertainty about actual load values, it is inadmissible to contemplate a partial factor of unity for normal loads. However, when considering accidental loadings the general level of uncertainty is much higher and values of 0.95 and 1.05 represent just token differences from unity.

The lower values assigned to dead, imposed and wind loads when acting in combination reflect an assessment of the reduced probability that extreme values would occur simultaneously. One of the notes in Table 1 refers to the alternative values to be considered for design wind load. These values correspond to the nominal horizontal load for which a building should be checked to provide resistance to overturning (see clause 20.1). In the context of the whole building this force is directly comparable to wind loading and in effect ensures that a building is designed to resist a minimum wind load. It is mainly when considering longitudinal stability of narrow buildings that 0.015 G_k will be the critical value of wind load.

These load combinations apply equally to individual walls and columns and in such cases 0.015 G_k as applied to the whole building should be distributed in the same way as wind loads would be through the building ie on the basis of relative stiffness, to determine the load on individual elements.

Although all combinations of loading should be examined to determine that which is the most severe it will often be possible to select the most onerous conditions based on experience.

Three particular conditions of dead and imposed load may be considered. Firstly, cases such as freestanding walls in which the weight of the wall is a major if not the only restoring force to resist overturning moments. Clearly, minimum values of vertical design load should apply and γ_f should be taken as 0.9. Secondly the combination $1.4G_k + 1.6\,Q_k$ will apply to internal walls to which vertical load is applied only by a floor spanning to one side of the wall. Thirdly, when floors span on to a wall loaded also from above it may be necessary to consider a range of combinations of dead and imposed load on each element. Although $1.4\,G_k + 1.6\,Q_k$ will give rise to the greatest total load on the wall, $0.9\,G_k + 1.6\,Q_k$ on a floor or from above may lead to a greater total eccentricity of a smaller load which may be a more onerous condition for the wall (see Figure 7(a). Guidance based on consideration of a wide range of possibilities is given in Chapter 3 of this handbook. A more onerous combination still may arise if Q_k is taken as zero on some elements. The code does not require this to be considered and the number of combinations would become impracticably large. Although CP 110 envisages this condition for a continuous frame the minimum partial safety factor on G_k is 1.0. The more logical use of 0.9 G_k in BS 5628 to some extent removes the need to consider zero imposed load.

When considering dead and wind load acting together it is necessary to use 0.9 G_k in cases when net tensile stresses could occur, such as in top floor walls or lightweight structures for which

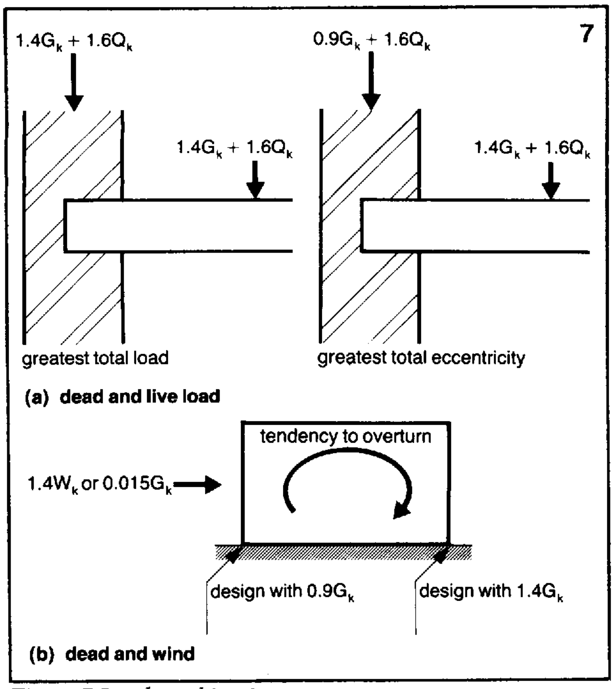


Figure 7 Load combinations.

the ratio of wind load to dead load is relatively high. The contrasting cases are shown in Figure 7(b).

The facility provided by the partial factor format to adjust the margin of safety for different parts of a structure is illustrated by a further alternative value of γ_f for certain types of element. Freestanding walls and laterally loaded panel walls which may be removed without affecting the stability of the overall building in any way may be designed for a dead and wind load combination of $1.4\ G_k + 1.2W_k$. Although the principle of considering varying degrees of reliability stands in its own right, the logic of the reduction was stimulated by the comparison between the experimental results and the design procedure given in clause 36 and by the difficulty of justifying the design of panel walls of apparently acceptable size. The reduction thus rests on a calibration against current practice.

23. Characteristic compressive strength of masonry, f_k

23.1 Normal masonry

Compressive strength of masonry, including that of the component structural units and mortar, has for long received the greatest attention in research and testing. Work has been necessary to increase the reliability of earlier measurements and to examine the effects of changes in type of brick and block and mortar which have resulted from innovation and other changes in practice. Therefore the revision of this code of practice has taken account of the latest experimental data in addition to the need to present the compressive strength in ultimate or characteristic terms.

As a result of this new data and re-appraisal there is not a simple correspondence between the values in BS 5628 and the basic permissible stresses in CP 111. In fact the ratio of stresses in code Table 2a to the corresponding stresses in CP 111 ranges from 4.04 to 5.24. However, the overall mean ratio is about 4.5 and for unit strengths less than or equal to 50 N/mm^2 the ratio is about 4.6, generally being higher for the lower unit strengths. These differences arise primarily because of new data on walls built with high strength bricks which suggest that earlier permissible stresses were perhaps too high. Equally some of the walls with low strength units, in which the mortar strength approached or exceeded the unit strength, were considered to have been underrated previously. A further result was that the ratio of wall strength to unit strength seemed to be more continuously variable so that the data were represented better by a smooth curve. This again explains the variations between the new and earlier recommendations. The relationship between these stress ratios, the safety factor implicit in CP 111 and the partial safety factors in BS 5628 is discussed in clause 27.

The averaging processes involved in reducing a large mass of data to simple relationships for general use inevitably produce values which for particular units may be conservative. If undue disadvantage is felt to accrue it is permissible to carry out wall tests to determine compressive strength. In some cases this procedure is essential (clause 23.1.4) and the test procedure is given in code Appendix A.2. It is worth remarking at this juncture that despite investigation of a wide range of shapes and sizes of masonry test specimens it is considered still in the UK that storey height wall testing gives the most realistic assessment of the likely performance of masonry in actual buildings and that, using soundly based tabulated data, compressive strength of masonry correlates best with compressive strength of units and of mortar.

The data given in code Table 2 relate to normal masonry, that is masonry in which the structural units are laid on their normal bed faces in the attitude in which their compressive strength is usually determined. Bonding is described only as normal which may be taken to imply that at least vertical joints must not be continuous. Clause 8 recognises that masonry units may be laid in other attitudes eg bricks laid on edge or rectangular section blocks laid flat, and that the bedded area of perforated bricks and hollow blocks is less than that of solid units. Clauses 23.2 and 23.3 place conditions on the use of the values in code Table 2.

There are a number of reasons why the compressive strength ie the mean strength of a sample tested according to the appropriate BS, has been retained for unit strength, rather than characteristic strength which might have seemed more appropriate in a semi-probabilistic approach. Firstly, the relevant existing standards for determining and controlling compressive strength specify mean values. Secondly, the characteristic value of a population is related directly to the mean strength for a given coefficient of variation, which in the long run is sensibly constant. Within limits, too, the strength of masonry obtained for a given unit is not sensitive to small variation in unit strength. However, it is important to realise that although unit strength, mortar designation and the methods of their determination are not necessarily unique considerations, the values and methods quoted in BS 5628 are those that relate specifically to the tabulated values of masonry strength as determined generally from wall tests. Unilateral alteration of one parameter, such as the method of testing bricks, could invalidate the tables in the absence of other correlations.

Code Table 2(a) applies to masonry built with bricks of standard format complying with BS 3921, BS 187 or BS 1180 and having a ratio of height to width of about 0.6. By implication from clause 23.1.2 the values apply to walls 215 mm or greater in thickness. Code Table 2(b) is a repetition of the lower strength range of code Table 2(a) with the unit strengths altered to accommodate commonly produced concrete blocks. This table has been provided so that masonry strengths for units having ratios of height to width between 0.6 and 2.0 may be obtained by interpolation with code Tables 2(c) and 2(d) – see clauses 23.1.5 and 23.1.6.

Code Tables 2(c) and 2(d) refer to hollow blocks and solid concrete blocks respectively whose ratio of height to width lies between 2.0 and 4.0 inclusive. It is important to note that these tables take account of the effect of the shape of a unit on its compressive strength so that for shapes in this range no further modification factor has to be applied for normal masonry, unlike the procedure in CP 111.

The compressive strength of the basic material from which bricks or blocks are made may be considered as an intrinsic property of that material. However, when attempts are made to measure that strength, differing apparent strengths are obtained depending on the shape of the test specimen used (see Figure 8). A squat, brick-shaped specimen will have a much higher apparent strength than a slender, block-shaped specimen fabricated from the same material. The difference is due primarily to the restraint of the loading plattens of the testing machine which has a smaller effect in the middle region of the more slender specimen. In the absence of other factors the resulting masonry strength is independent of the shape of the units for the same basic material. However in practice, taller units for a given width reduce the number of mortar joints in a given height so that the overall effect of unit shape on masonry strength is more complex. The detailed implications of the factors involved have not been studied extensively but the overall effect is shown by the difference between code Tables 2(b) and 2(d). Slender units (aspect ratio between 2.0 and 4.0) of the same measured ie apparent, strength as squat units (aspect ratio 0.6) of a different

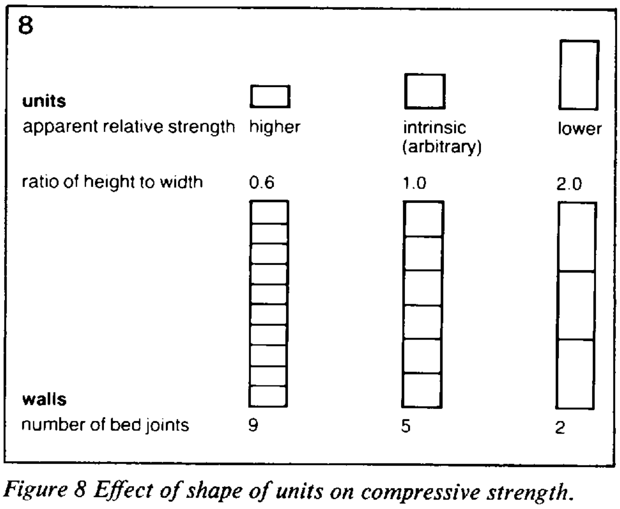


Figure 8 Effect of shape of units on compressive strength.

material can be seen to provide walls of double the strength. Conversely, however, because of the effect of joints, a squat unit cut from a slender unit might have double the strength of the slender unit but might produce masonry of rather less than double the strength.

In code Tables 2(c) and 2(d) it can be seen that lower strength slender units can produce masonry of the same characteristic strength as the mean strength of the units, that is mean masonry strength higher than mean unit strength. Although somewhat surprising at first sight, it is explained by several factors. At these low strengths, mortar strength is similar to unit strength so that the masonry is more homogeneous. Also, the enhancement of unit strength provided by confinement in the wall is relatively more important at these strengths. It is frequently found that the strengths determined for dry blocks are higher than when the blocks are wet. BS 2028, 1364 requires blocks to be tested wet, the mean strength so determined being used in code Tables 2(b), (c) and (d), but the wall tests are carried out dry. Masonry constructed from slender hollow blocks does not show the same enhancement, see code Table 2(c), because the constraints of adjacent units are less effective in preventing failure.

In all four tables it is permissible to interpolate between values of characteristic strength for intermediate values of unit strength. This interpolation is facilitated by code Figures 1(a), (b), (c) and (d). They also demonstrate graphically the point that the ratio of masonry to unit strength decreases as unit strength increases and also the relatively limited effect of mortar designation on masonry strength. The following clauses describe circumstances under which modifications should be made to the tabulated values.

23.1.1 Walls or columns of small plan area

The random variation of unit strength about the mean value is normally taken account of in the quoted value for masonry strength. However, in small samples of masonry there is an increased possibility that sufficient low strength units may be present for the strength of a wall to be affected adversely. Although this effect is statistical and could have been accommodated by a partial safety factor it is essentially a geometrical effect on compressive strength and has been retained as a modification to masonry strength in line with CP 111. In the absence of adverse experience with the previous reduction factor the maximum area of application has been relaxed slightly but the progressive effect has been strengthened because walls of very small plan area are likely to be particularly sensitive to the effects of slenderness and eccentric loads.

23.1.2 Narrow brick walls

Construction of walls of width equal to or greater than the length of the units might appear to afford even greater restraint to individual bricks so enhancing the wall strength. In practice, however, compressive failure occurs predominantly by the formation of vertical cracks. Greater resistance to their development is afforded by the continuous brick cross-section of a narrow (half brick) wall than by a wider section which is weakened by the presence of more vertical mortar joints. Test results support the use of an enhancement factor of 1.15 which may be applied to half brick walls made from British Standard bricks. Amendment No. 2 extends the enhancement to cavity walls.

23.1.3 Walls constructed in modular bricks
Modular bricks 90mm wide × 90mm high are more slender than the standard format bricks which form the basis of code Table 2(a), so that taking account of the effect of shape as discussed above a somewhat higher masonry strength should be expected. On the basis of limited experimental data a 10% enhancement of the values in code Table 2(a) is suggested. When the masonry thickness equals the brick width the enhancement factor is 1.25, corresponding to the 15% enhancement for narrow bricks walls applied in addition to the 10% for modular bricks.

23.1.4 Walls constructed of wide bricks
Following the discussion above on the effect of shape of unit it is clear that such bricks are more squat than those covered by code Table 2(a) and require special consideration. Information is not generally available for this shape of unit but once obtained in accordance with Appendix A.2 no further testing is required for similar conditions of use; however, many manufacturers of such bricks will have test data available for designers, and should be consulted.

23.1.5 Hollow block walls
Code Table 2(b) is provided so that the strength of walls built of hollow blocks intermediate in aspect ratio between 0.6 and 2.0 may be obtained by interpolation with code Table 2(c). It applies to clay and concrete blocks which contain holes or cavities.

It is important to bond the units in a pattern which ensures that the webs are aligned vertically.

23.1.6 Solid concrete block walls
Interpolation between code Table 2(b) and Table 2(d) enables the strength of walls built of solid concrete blocks intermediate in aspect ratio between 0.6 and 2.0 to be obtained. Concrete blocks in this case must be solid, having no perforations or cavities, whether or not they pass right through the block. This requirement is more restrictive than the definition in BS 2028 which permits a solid block to have up to 25% voids.

23.1.7 Walls of hollow concrete blocks filled with in situ concrete
If the block strengths are obtained by dividing the failing loads in tests by the actual cross-section of the concrete webs carrying the load, contrary to the procedure required by BS 2028, 1364, a higher value of strength is obtained. This increase is an approximate measure of the effect of filling the cavities of a block with concrete of strength at least as high as the strength of the blocks, as calculated by this modified, net area, method. This higher strength may be used in or, for interpolation between, code Tables 2(b) and 2(d).

23.1.8 Natural stone masonry
No new data of general relevance are available for stone masonry and the recommendations follow those in CP 111. The clause recognises the reduced effect of mortar joints on stone masonry strength when the joints are relatively widely spaced and are relatively thin as a result of careful preparation of the mating surfaces. Any increase over the recommendations for characteristic strength for solid concrete blocks must be based on the designer's experience and judgment or suitable testing.

23.1.9 Random rubble masonry
As noted under clause 3.20 random rubble is not covered strictly by the term 'structural units'. However this recommendation repeats that contained in CP 111 in the absence of any alternative guidance.

23.2 Structural units laid other than on the normal bed face
Clause 8.1 has indicated already that the strength is applicable only to the direction in which the unit was tested. This clause emphasises that the appropriate strength should be used in code Table 2.

23.3 Perforated bricks and hollow blocks
This clause notes that the strength of units which are not solid is determined also by dividing their failure load by the gross plan area of the unit. When hollow blocks are bedded on mortar strips along only the outer edges load is transmitted only through these strips. The strength of the wall is affected and the value determined from code Table 2 should be reduced by the ratio of the bedded area to the net plan area of the block, taking into account the perforations, unless the block has been tested on two strips of mortar in accordance with clause 31.2.4 of BS 3921.

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C&CA Report 42.467, 1972.

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Proc Brit Ceram Soc April 1973 pp 79–98.

Beech D G and West H W H,
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A supplementary literature list relating to the properties and use of concrete blocks.
C&CA Report 42.495, 1974.

24. Characteristic flexural strength of masonry, f_{kx}

CP 111 suggests values for allowable tensile stresses in bending to be used at the designer's discretion. Since that code excludes consideration of walls carrying only self-weight and wind load it is unclear under what circumstances the discretion is to be exercised, other than to resist uplift, for example, due to wind suction on roofs. Now, however, the inclusion of design information for such walls, defined as laterally loaded wall panels, is a major advance in the new code.

The design methods given in 36.4 (and 36.5 for free-standing walls) rely on a knowledge of the flexural strength of masonry. The word 'flexural' is used deliberately to emphasise that this strength parameter, although substantially tensile by nature, is based on data obtained from tests carried out in bending, or flexure. Thus the same caveat applies as before to using masonry in direct tension: it is left to the designer's discretion (see below).

Attention has been drawn already to the two principal directions of flexural failure of masonry in the explanation of the term orthogonal ratio. They are depicted in code Table 3 in which the failure surface is called a plane. For failure in the weaker direction, along the bedding, this description is reasonable. The nature of the failure surface in the orthogonal direction is more complicated. Again as shown in the code, failure may cause a toothed or interlocking surface to develop in which flexural tension develops across the perpends and rotational shear occurs in the bed joints. However, when mortar and unit strengths are closer to each other, flexural tension may develop across the whole section causing a more or less plane fracture.

The values quoted represent a significant amount of experimental data. Most of the testing has been carried out by the British Ceramic Research Association, part of the programme being funded by BRE. In the course of the work a range of parameters including water absorption, initial rate of suction and unit flexural strength were investigated for a possible correlation with flexural strength of masonry. The most significant correlation for clay bricks was obtained with water absorption and the selection of three ranges takes optimum advantage of the overall spread of data (see Figure 9). For concrete blocks, unit compressive strength correlated well for the stronger direction but in the weaker direction there was general scatter but little correlation with strength of block or any other property.

The test procedure described in Appendix A.3 is basically that used by the British Ceramic Research Association. The specimen size and loading arrangement have been determined from the need to incorporate as many joints as possible in the loaded region, the largest specimen conveniently movable in the laboratory and provision of as large a zone as possible with a uniform bending moment by limitation of the shear span ratio. Various research workers have used smaller specimens and different loading arrangements but by suitable transformation the stresses at failure in the different tests may be compared with each other. Although no standardised test method has been agreed yet, that proposed here represents an optimisation of the various constraints. An appraisal of the method is being carried out at BRE.

Measurements of flexural strength are characterised by a higher variability than found for compressive strength. Calculation of the characteristic value takes this into account but when the coefficient of variation is high, say above 15%, the normal distribution becomes inappropriate and Appendix A.3 proposes the use of the log-normal distribution as discussed under section 19. Negligible error accrues from using a log-normal distribution when the standard deviation is small.

Appendix A.3 recognises variation in width of masonry but most of the experimental data refers to 102.5 mm bricks and 100 mm blocks. There are some indications that thicker wallettes may have lower strengths but the code draws no distinctions. Equally, the moisture content of units at the time of laying has been considered of some relevance. The bricklayer may certainly wish to adjust the suction of bricks by 'docking', as recommended for example by CP 121, to facilitate laying. Although an improved bond may result, the flexural strength of wallettes built from a range of undocked bricks having high water absorption exceeded the minimum values quoted in the code (ie, for water absorption over 12%).

The contrast between the permissible values quoted in CP 111 and the new values is marked. In the perpendicular direction, for mortar designations higher than (iv), most characteristic

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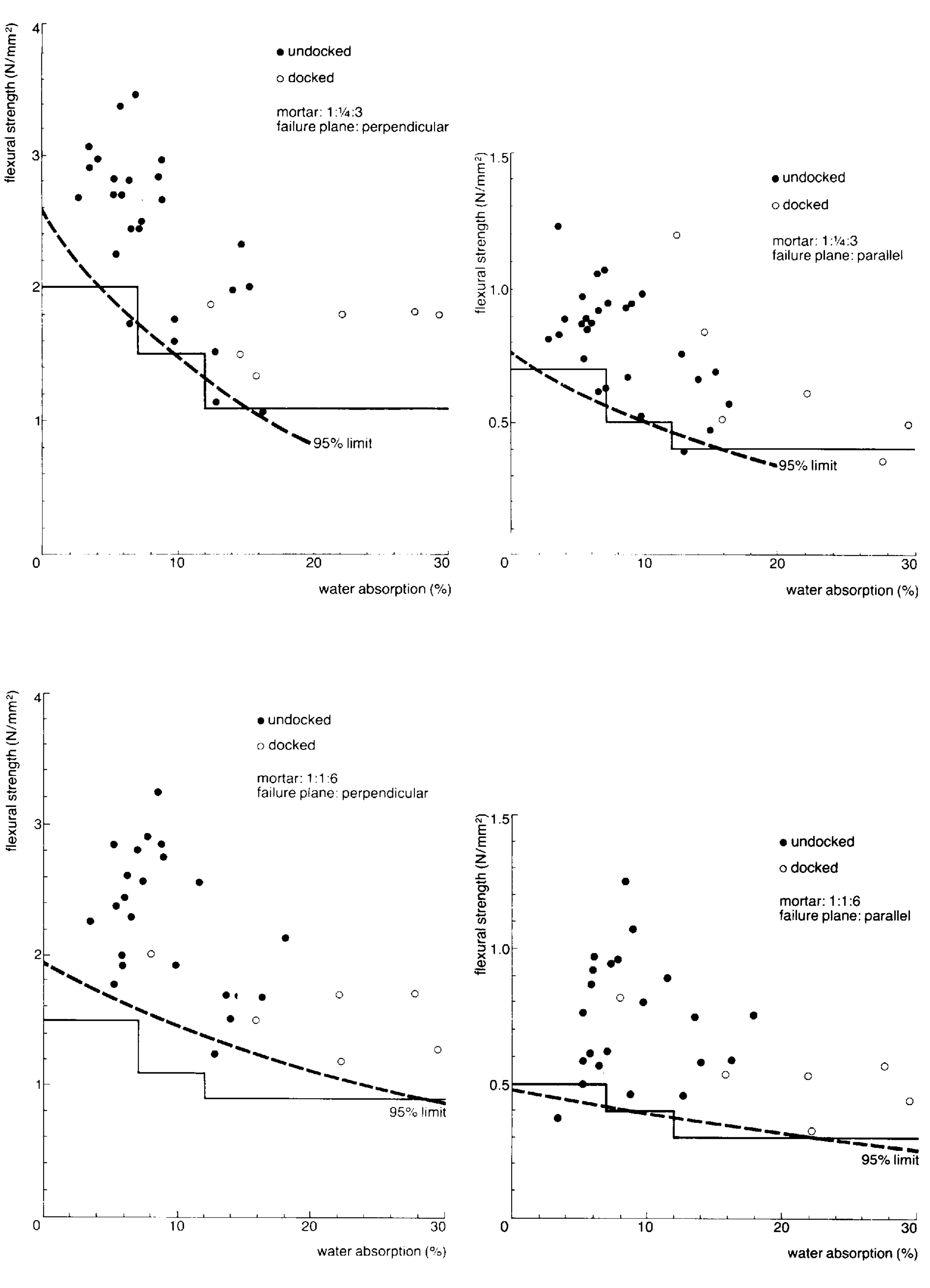

Figure 9 Flexural strengths and water absorption.

values of f_{kx} are greater than before, except for lower strength concrete blocks which were excluded previously, anyway. However, generally only clay bricks with water absorption lower than 12% have strengths in the parallel direction greater than before. In the design procedure described later it will be seen that use is made of the higher (perpendicular) value of f_{kx} in conjunction with the orthogonal ratio, not involving explicitly f_{kx} in the parallel direction, other than for walls spanning in a vertical direction only.

The three explicit conditions governing the use of flexural strength relate to direct tension, flexure at a damp proof course, and limitation of

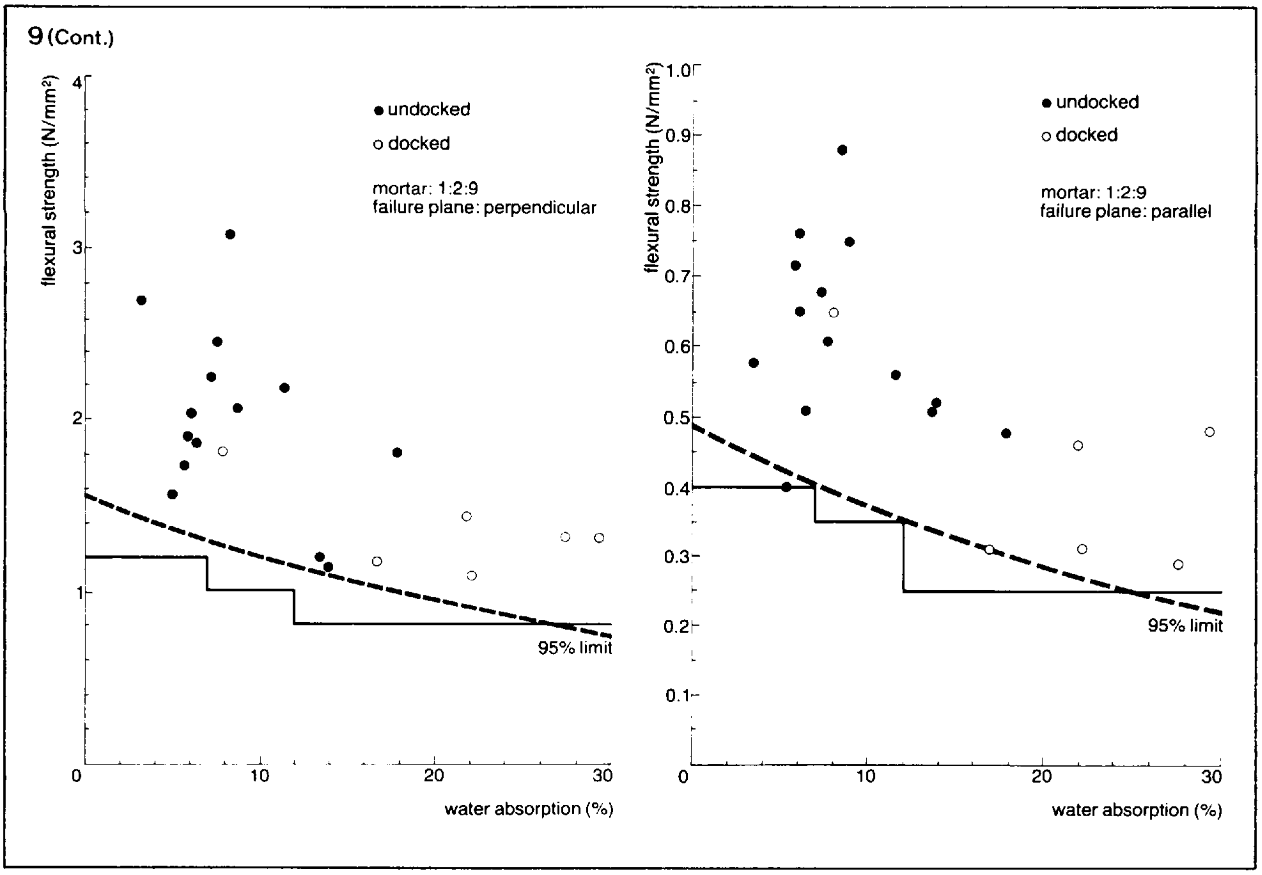

Figure 9 Flexural strengths and water absorption.

combined flexural and direct tensile stresses. It is not generally acceptable to rely on masonry in direct tension but, under some circumstances it may be permissible, such as resisting wind uplift on roofs or accidental loads. The direct tensile stresses should be limited to half the flexural strengths in code Table 3.

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The Structural Engineer v 54 n 7 239–246 1976.

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The Structural Engineer v 55 n 10 1977.

25. Characteristic shear strength of masonry, f_v
There are several types of shear failure of masonry. Vertical shear may occur, particularly at the junction of intersecting walls, in which masonry units bonding the walls together will suffer shear failure. Horizontal shear may occur along bedding surfaces, particularly at the level of damp-proof membranes. However, unless the wall is particularly long or the vertical load sufficiently high, failure by horizontal in-plane forces is more likely to occur by rotation of the wall, effectively tensile or peeling failure of the mortar joints.

Once masonry is enclosed in some kind of bounding frame which offers restraint to the edges of the panel failure will occur by diagonal shear. Both diagonal and horizontal shear resistance are dependent on vertical stress in the masonry and the recommendations in the code relate to these conditions. If resistance to vertical shear is required it will be necessary to obtain experimental data.

The recommendations do not represent new data but are a translation into limit state terms of the values given in CP 111: 1970, as amended in 1971, with the addition of lower values for mortar designation (iv). The amendment was based largely on a reassessment of existing and additional experimental work. Further work has provided the addition for weaker mortar. It may be observed here that the convention for subscripts has lapsed as characteristic shear strength might have been expected to be denoted by f_{kv}.

The original relationship in CP 111 was effectively $0.1+0.16\ g_A\ N/mm^2$. A global safety factor of 3.5, equal to $\gamma_f=1.4$ multiplied by $\gamma_{mv}=2.5$ (see clause 27.4) transforms this permissible strength to a characteristic ultimate strength of $0.35+0.56\ g_A$, with a maximum of $0.5\times3.5=1.75\ N/mm^2$

If, however, the shear strength is required to resist wind load on a panel to which $\gamma_f=1.2$ is applicable, the horizontal shear capacity is effectively increased by 16% although the probability of failure is also increased. Care should be taken to choose the appropriate partial factors when determining design vertical load. The shear resistance of masonry will be required usually to resist wind loading and load combinations (b) or (c) in clause 22 could apply for normal loads. But clearly case (b) is the more critical because the design dead load will be lower and the design wind load higher. Normally, there will not be circumstances in which vertical imposed load can be included in g_A (see code Amendment No. 2).

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Proc 2nd Int Brick Masonry Conf. Stoke-on-Trent 1970. Ed West H W H and Speed K H BCeramRA 1971.

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Proc 2nd Int Brick Masonry Conf. Stoke-on-Trent 1970. Ed West H W H and Speed K H BCeramRA 1971.

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Proc 2nd Int Brick Masonry Conf Stoke-on-Trent 1970. Ed West H W H and Speed K H BCeramRA 1971.

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Shear behaviour of masonry walls subdivided by floorplates.
1st Canadian Masonry Symp Calgary 1976.

26. Coefficient of friction

This value has been based on limited work but seems to conform with values given in other national codes for masonry structures. Its value is identical to the slope of the shear strength/vertical load relationship in clause 25 for masonry but with zero value in the absence of vertical load. If a strong shear connection is required masonry will offer greater resistance than can be provided by friction of, for example, a concrete floor bearing on a wall. Conversely, being weaker, such a joint may represent the critical limit state under certain load combinations. The main use of friction probably lies in design to resist accidental damage.

27. Partial safety factors for material strength γ_m

27.1 General

Consideration of the values for γ_m, the partial safety factor for material strength, represents the essence of limit state design, particularly as applied to masonry. The possible effects covered by γ_f have been considered under clause 19 and it has been shown that, ideally at least, γ_f should be independent of the structural material and relate only to the applied loading regime. γ_m must, of necessity, take account of the differing characteristics of the various structural materials.

It may be helpful to suggest an alternative framework within which to consider clause 27, since the relation to quality control recommended in clause 27.1 does not apply to all values of γ_m. Firstly, distinctions may be drawn between limit states for each material, ie, compressive and flexural failure (27.3) and shear failure (27.4), and finally account may be taken of abnormal loading (27.3, 27.4 and 27.5).

The requirements for quality control (27.2) have been omitted from this framework deliberately, not because their relevance and importance should be minimised in any way, but because they represent essentially a separate set of conditions which could be applied to all aspects of the framework. However, because the compressive and flexural strengths of masonry are usually the most important strength parameters, the question of quality control has been applied only to them, and the clause built up around this aspect. Improvements in quality of these parameters are also likely to offer the best return to the designer in terms of economy and lead to better quality of other properties.

27.2 Quality control

27.3 Values of γ_m for normal and accidental loads

27.4 Values of γ_{mv} for shear loads

27.5 Values of γ_m for use with ties

In CP 110, only single values are recommended under normal loads for each type of γ_m, and steel and concrete strengths are the only relevant parameters. However, it could be considered that the differences between 'designed' and 'prescribed' mixes are tantamount to two levels of quality control for concrete strength. In this code, the flexibility presented by the partial factor format has been used as positive encouragement to the production of masonry of improved quality. The two most significant factors influencing the compressive strength of masonry are the compressive strength of units and the manner in which the units are put together on site to form the masonry. Therefore two types of quality control are conceived, based on control of strength – 'manufacturing' control (clause 27.2.1) – and on control of workmanship and mortar – 'construction' control (clause 27.2.2). For each type of control 'normal' and 'special' categories are defined.

The difficulties in applying this approach have centred essentially on determining values for γ_m which, in relation to each other, are justified by the differences achieved by the control methods, and which, in relation to overall safety, represent realistic levels of reliability. As indicated above under clause 19, it is difficult in practice to establish analytically partial factors which relate the overall reliability of a masonry element to the variability of the influences on it. Thus γ_m has to be judged generally against established practice, an act of calibration, and the partition of γ_m between the combinations of categories of control may be assessed analytically to judge relative values.

In translating basic compressive stresses in CP 111 into characteristic values it was assumed that CP 111 embodied a safety factor of at least 5. This is the load factor recommended in CP 111 for tests on squat walls. As it related to basic stresses which were already tending to lower limit values it was considered that 4.8 would be a reasonable factor of safety in relation to characteristic compressive values. Taking a typical partial safety factor for loads as 1.5 leads to a partial safety factor for materials of 3.2. This value should represent existing practice which must in reality include a range of construction in which quality control, particularly of structural units, is both better and worse than average. It was then envisaged that two categories of manufacturing control could be established as equal margins above and below the nominal value of 3.2 Any numbers must be construed as indicative of general arguments which although broadly correct cannot be exactly so. In particular the value of γ_f overall depends on the ratio of dead to live load which will vary. Retaining these arguments, the value of 3.2 was modified to 3.3 (the mean of 3.1 and 3.5), and the other categories of control related to it. When dead load predominates, $\gamma_f = 1.4$ and the overall factor of safety is $1.4 \times 3.3 = 4.6$ which accords with the overall relationship discussed in clause 23 between f_k and basic stress in CP 111.

In relation to the highest value of $\gamma_m = 3.5$, the other lower values of γ_m represent a judgment of the improvements in strength which may be attributed to improved manufacturing control, viz an improvement of about 12%, by applying clause 27.2.1.2, and to improved construction control, viz an improvement of about 25%, by applying clause 27.2.2.2. Both enhancements may be applied independently of each other if justified.

For bricks, the use of quality control schemes to demonstrate the attainment of an 'acceptance' limit is often practised already by manufacturers. The aim of a better guarantee of strength is sound because the average strength of bricks is more important in determining masonry strength than the individual or minimum strength. The reliability attached to the value of an 'acceptance' limit is reflected reasonably in the 12% enhancement. Many manufacturers operate suitable control schemes, and the Calcium Silicate Brick Manufacturers Association has recommended appropriate quality control procedures to its members.

It is to be expected that advantage will be taken frequently of $\gamma_m = 3.1$, as long as manufacturers provide the necessary information. It should be understood that manufacturing control implies a more reliable estimate of the strength of the bricks being produced, and does not imply that the manufacturing process itself is being altered so as to produce a more reliable product. It is worth noting that 'special category of manufacturing control' should not be confused with 'special quality' bricks which refers to their durability as defined in BS 3921.

The enhancement afforded by special category construction control must be acknowledged to represent a more subjective assessment of the benefits to be derived from improved control of site workmanship and mortar strength. It recognises the costs involved in providing the necessary supervision and testing, and balances them against the economic advantages to be gained from the enhanced strength. Clearly, the application of these procedures will be justified only on major works, but factors which have been shown to have a significant effect on compressive strength include failure to fill bed joints, bed joints of excessive thickness and unfavourable curing conditions, the latter also affecting bond (flexural) strength.

Before examining other values for γ_m it should be noted that the values given in code Table 4 are to be applied to flexural strength f_{kx} as well as compressive strength f_k. The data supporting the effect of the quality control measures on flexural strength is not substantial. However, the design predictions based on clause 36.4 using the lowest value of $\gamma_m = 2.5$ compare favourably with the experimental strengths of laterally loaded panels. In view of the importance of using the correct value of f_{kx} appropriate to the water absorption, it is recommended, particularly when using reduced values of γ_m, that manufacturers' assurances about the value of water absorption should be obtained. In general terms there is a loose correlation between the water absorption of clay bricks and their compressive strength, so that better assessment of the latter should ensure greater reliability of the former. With concrete blockwork, there is a broad correlation between the compressive strength of blocks and the flexural strength of blocks, and between the flexural strength of blocks and the flexural strength of blockwork. Again, control of compressive strength will tend to better reliability of f_{kx}, particularly for the perpendicular direction used for most panel wall design. As there is no obvious correlation in the weaker parallel direction reduced values of γ_m should be considered carefully. Values of f_{kx} in this direction are used for vertically spanning walls and, suitably reduced as recommended in clause 24.1, for resisting wind uplift forces.

The option of testing masonry specimens to obtain strength data, as an alternative to using tabulated values, is encouraged throughout the code. Tabulated values are of more general application and by incorporating a wider range of materials are necessarily more conservative than information obtained from and restricted to only one material. This more specific data often implies the use of a higher characteristic value than tabulated, and in recognition of the improved information a 10% reduction of the partial factors in code Table 4 is permitted. An inseparable consequence of this allowance is that it is restricted to the use of the circumstances of the test.

The single partial factor for shear of masonry, γ_{mv}, derives from the global factor implicit in CP 111 and represents the relatively lower variability obtained in experiments on specimens. The value for ties represents an allowance for possible shortcomings in the quality of installation including poor bedding and lack of normality to plane of wall.

When considering the effects of abnormal loads, about whose magnitude there is considerable uncertainty, the reduction in reliability by halving all the values assumed for γ_m in the design for normal loads is equivalent to increasing the probability of failure by something like two orders of magnitude. The exception to this reduction applies to the lateral strength of axially loaded walls and columns (36.8) for which γ_m included in the design procedure is a general safety factor, not specifically related to material strength (see also discussion under clause 19).

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The effect of site factors on masonry performance.
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Model specification for loadbearing clay brickwork.
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SECTION 4: DESIGN: DETAILED CONSIDERATIONS

The code has provided in the first three sections a framework of basic conditions within which the design of individual masonry elements has to be performed. Thus, within a discussion of overall structural performance and of the philosophy of design to limit states, guidance has been given to enable the loading regimes to be established and to assess the inherent strengths of the available materials. The next stage in the design is to establish the local constraints on elements as incorporated in the structure so that design strengths or resistances may be established and equated to design loads or moments. This Section therefore suggests design procedures for the full range of combinations of applied vertical and horizontal loads, both normal to and in the plane of the wall, and suggests the degrees of restraint and resistance which may be provided by various other structural elements or within the masonry itself to enhance the strength of a simple unsupported masonry element.

Therefore, the first five clauses 28, 29, 30, 31 and 32 deal primarily with aspects of vertical load carrying capacity of walls together with those cases in which lateral load may be treated as an eccentricity of vertical load. The next three clauses deal with types of loading which are generally less critical but nevertheless of significance: shear (33), concentrated loads (34) and composite action with supporting beams (35). The final clause (36) deals with all other aspects of walls subjected to lateral loads.

Consideration of slenderness of walls and columns

28.1 Slenderness ratio

The definition of slenderness ratio given in clause 3 requires the determination of effective height, length and thickness of walls. Guidance is given in clauses 28.3 and 28.4, the recommendations in the former clause being based on the conditions of restraint discussed in 28.2 and illustrated in Appendix C. It should be noted that the slenderness ratio to be used in code Table 7 and Appendix B is based on whichever is the lesser of effective height and effective length.

As the slenderness of an axially loaded member increases its load carrying capacity reduces as its tendency towards buckling failure increases. In order to avoid the possibility of sudden buckling failure, without warning, a limiting slenderness ratio of 27 is adopted which is independent of the load to be carried and is simply a function of the geometry of the wall. The value is not derived analytically but based on judgment and assessment of experimental tests on walls. The limit is the same as that incorporated in the 1971 amendment of CP 111: Part 2: 1970 and represents a slight relaxation of the more stringent limits which had been introduced earlier, although part of the change was due simply to the reduction from nominal to actual thickness. At these high slenderness ratios the load carrying capacity is so reduced anyway that a practical limit would be placed on the usefulness of even more slender walls. The practical limitations on workmanship and accuracy of construction inevitable when using very thin units (ie less than 90 mm) in buildings higher than two storeys (ie more than typical houses) lead to the limitation of slenderness ratio to 20.

28.2 Lateral support

Clause 20 drew attention to the need to ensure that all masonry structures are adequately braced both to resist horizontal loads and to prevent sidesway of walls, the liability to buckling of which would otherwise be considerably increased. It is normally convenient to provide this resistance by lateral supports which act either in a horizontal plane or a vertical plane. As far as the supported wall is concerned its interaction with these planes is either a horizontal or a vertical line. This clause provides the strength criteria for lateral supports (28.2.1) and indicates the restraint provided by typical forms of construction to the supported wall or column (28.2.2 and 28.2.3).

28.2.1 Horizontal or vertical lateral supports

The two load bearing requirements for lateral supports have been contained in CP 111 since its two earliest versions. Statement (a) that 'the simple static reactions to the total applied design horizontal forces at the line of lateral support' shall be resisted is self-explanatory. Statement (b) derives from consideration of the horizontal forces required to stabilise a wall out of plumb by a nominal amount of $\tan^{-1} 0.025$ or 6 mm in a storey height of 2.5 m. Field measurements of the accuracies achieved in construction confirm this assumption. These two requirements apply to individual walls and columns and their lateral supports. When walls or other components are provided to resist the total horizontal forces on the structure as opposed to individual elements the out of plumbness consideration may be ignored on the basis that it is catered for separately. It is equally important to consider and design the connections between walls, supports and bracing, whether for elements or the whole structure, to transmit the same loads as the supports etc have been designed to resist. In other words, the possibility of overlooking connections should be carefully avoided to ensure that they do not become the weak link in the overall structural concept.

In general terms, the greater the restraint applied to the edges of a masonry wall or column, the greater the load it can carry. Restraint may be either resistance to movement laterally ie normal to the plane of the wall, or resistance to rotation of the edges of the wall. Similar considerations apply to the ends of a column, but along both its horizontal axes of symmetry. These concepts of translational and rotational stiffness are well established in structural analysis. It should be noted in particular that resistance to translation implies fundamentally a stiffness of support, although it may be convenient to interpret it in practice as a force. Resistance to rotation does not necessarily imply no rotation. When it is possible to take into account the relative stiffnesses of elements it may be possible to calculate directly the effective height or length of walls and columns without the need to use the guidance given in 28.2.2, 28.2.3 and 28.3. Based on experience, however, these clauses together with Appendix C show the restraint offered by various common forms of construction for which nominal values for effective dimensions may be used. Since the resistances afforded are only nominal, but adequate, forms of idealised restraints they have been distinguished by the descriptions 'simple resistance' (cf translational restraint) and 'enhanced resistance' (cf rotational restraint).

28.2.2 Horizontal lateral supports

The array of conditions in this clause may be summarised as follows:

(a) Simple resistance is provided by all types of floors or roofs which abut walls as long as straps are provided when required, as indicated in Appendix C. Exceptions to the need for straps are concrete floors abutting an internal wall on both sides at the same level (code Figures 22, 23 and 24) and timber joists, in houses of not more than three storeys, provided that they are separated by not more than 1.2 m and bearing on not less than 90 mm of wall or are supported on typical joist hangers. As an alternative to straps for timber joists a stronger type of joist hanger, as illustrated in code Figures 13 and 14, may be used (code Figures 12 and 19). The recommendations on the spacing of straps and anchors are contained in appendix C. The intervals should not exceed 1.20m (page 31 of the code is incorrect) except in houses of not more than three storeys when they may be increased to 2m.

(b) No types of floor or roof can provide enhanced support unless they span across, or onto by at least 90 mm, their supporting wall. The application to timber floors spanning from one side only is restricted to houses of not more than three storeys. In higher buildings enhanced resistance will require floors of a greater stiffness than can be provided by timber. The exceptions to the general requirements therefore apply to housing and connections to internal walls; examples are shown in Figure 10. The code of necessity refers to 'typical' joist hangers, meaning as commonly available, because there is no relevant standard. One of the main objectives of the design of hanger illustrated in code Figures 13 and 14 is to provide an improved tensile connection to masonry.

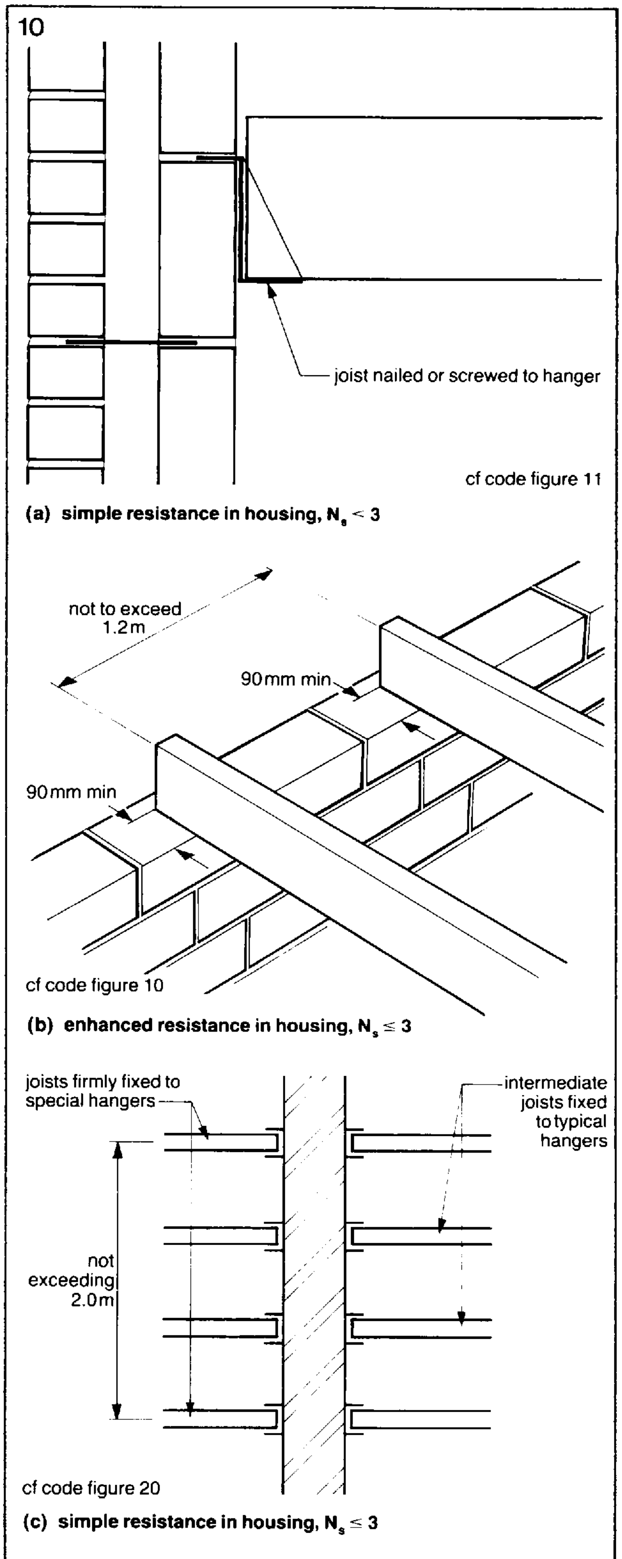


Figure 10 Horizontal lateral supports not requiring straps.

It has been remarked already that the key feature for mobilising lateral support is an adequate connection which introduces no additional flexibility or degrees of freedom into the interface between walls and lateral supports. Particular points to note are setting joist hangers tightly against the supported wall and securing the joists firmly to the hangers, adequate packing between wall and joists abutting it and adequate filling between wall and precast concrete units if not tight against the wall. The design of connections usually requires closer attention when the floor units span in the direction parallel to the wall ie abut the wall, as there will be no connection at all unless deliberately provided. A particular difficulty to be considered in the detail is possible deflection of the floor. Although timber floors may be more flexible than concrete, particularly in housing, the ability of concrete floors to span greater distances conveniently may also incur increased deflection.

A number of general points about Appendix C may be noted here for convenience. The suggested connections do not constitute necessarily the only forms of suitable connections for lateral supports. They are based, however, on a widespread survey of current practice and therefore represent a sound cross-section of perceived good practice. Nevertheless there has been little experimental investigation of the behaviour of connections either as components on their own, where appropriate, or as parts of sub-assemblies comprising wall, connection and floor or roof. The research programme of BRE is planned to investigate this area which lacks quantification.

When designing straps or anchors it is important to design adequate fixings between strap and floor and between strap and wall. Mild steel will usually be suitable and a partial safety factor of $\gamma_m = 1.15$ should be applied to the appropriate characteristic strength. A more detailed discussion of the design of anchors may be necessary but the metal should be corrosion resistant and the durability of galvanised steel should be assessed if the straps or anchors are not likely to remain in a dry environment. Leaking flat roofs causing corrosion may have serious structural consequences beyond that of simple dampness within the building.

It is noted that the same details are generally applicable to roofs as well as floors even though the figures illustrate the latter. However, roofs may be subject to uplift forces as well and the need for ties to resist them must be considered. Guidance is not given because it is not relevant to a roof's function as a lateral support. A very common form of roofing is provided by trussed rafters which will be required usually to provide lateral support both to flank, ie front and back, and gable walls. In the case of gables, this support may be required at joist level as well as by the rafters to the verge. Similar considerations apply to the design of connections between the end trusses and gable walls, as well as between trusses and wall plate, and wall plate and wall. Simple resistance may be assumed. Guidance on connections is also available in the context of Schedule 7 of the Building Regulations. It should not be forgotten that in providing lateral support to gable walls almost certainly the resistance to the applied loads will have to be transmitted to shear walls by diagonal bracing in the roof.

28.2.3 Vertical lateral supports

The same concept of simple and enhanced resistance applies to vertical lateral supports, that is buttressing walls may provide resistance only to lateral movement or may in addition provide moment restraint at vertical edges of a wall. The significance of these restraints is discussed in 28.3.2 but the distinction is made here primarily between a return wall bonded to the supported wall (enhanced resistance) and two walls tied together (simple resistance).

Here again the recommendations are based more on experience than experimental quantification. Although not stated explicitly in clause 28.2.3.2, the length of return necessary to develop reasonable moment restraint should also be not less than ten times the thickness of the wall it restrains. Although shear in the bed joints may be the limiting factor for both types of restraint, enhanced resistance involves a more complicated state of rotational shear as well.

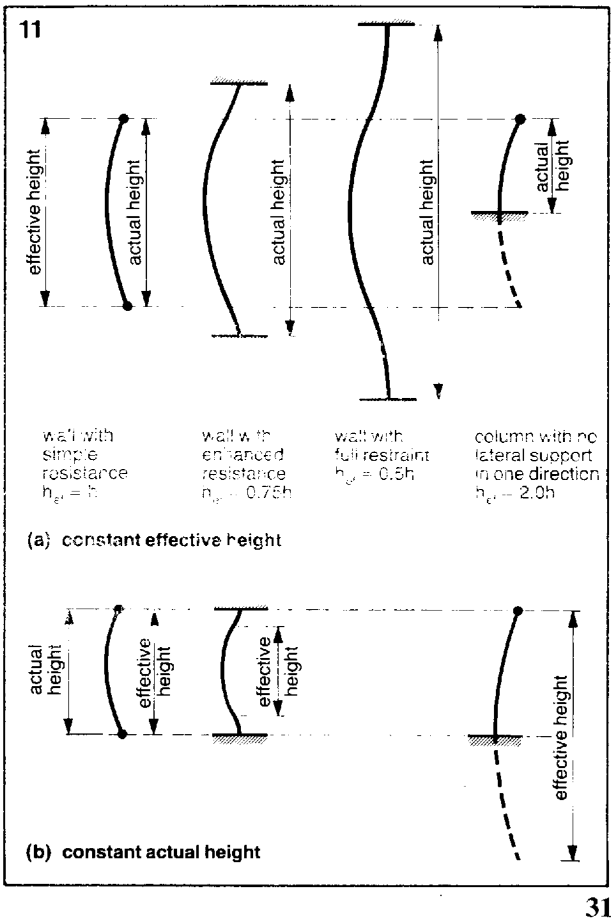
Again, other forms of construction eg columns, may be suitable (clause 28.2.3.3) provided that their adequacy can be justified. Although it may be possible to calculate the simple resistance provided by an alternative form, the ability to resist rotation is likely to be assessed subjectively.

28.3 Effective height or length

28.3.1 Effective height

The effective height is a measure of the relative susceptibility of a wall or column to buckling failure based on classical Euler buckling theory. An effective height equal to the clear height (28.3.1.1(b)) of a member having lateral supports providing simple resistance (28.2.2.1) corresponds to the length of a pin-ended Euler strut. The effect of restraint at the ends is to restrict the rotation and so increase the buckling load. This load may be equated to the strength of an effectively equivalent, shorter, pin-ended strut (Figure 11). That shorter length is equal to the length of the restrained strut between its points of contraflexure. Full restraint ie no rotation at the

Figure 11 Comparison of effective and actual heights for varions end conditions.



ends of a wall would provide an effective length $0.5 \times$ the clear height of the wall.

The factor 0.75 in 28.3.1.1(a) for walls restrained by one of the methods described in 28.2.2.2 is semi-empirical, based on experience and measurement of the behaviour of typical walls. Although it is not necessarily correct to make this global assumption it will generally be conservative to do so. Theoretical and experimental studies have shown that the actual effective height of a wall may be strongly influenced by the level and position at which it occurs in a building, particularly one having more than two or three storeys, and by the pattern of loading. Thus some internal walls near the base of a high building ie having substantial precompression, may have significantly higher restraint with a factor of 0.6 being appropriate. More commonly, perhaps, walls will be contained between floor and roof offering enhanced and simple resistance respectively. Then it may be appropriate to calculate an effective height between 0.75 and $1.0 \times$ the clear height. Common combinations of lateral supports are shown in Figure 12 with appropriate values for effective height.

Columns (28.3.1.2) have been shown by experiment to have a somewhat lower strength for a given height than have walls, mainly due to lower in-plane restraint. The effect of slenderness on the strength of a column is greater than for a comparable wall and an allowance for this is made by increasing the effective height to equal the clear height. It is assumed that columns will be provided at both ends with only translational restraint. If restraint is lacking at the top in one direction the effective height is double the actual height (Figure 11). Vertical load bearing walls, of course, should not be unrestrained in this way.

The method of treating columns formed between openings in walls is described in 28.3.2.3 and represents a presentation of earlier recommendations more explicitly in line with the types of lateral support given in 28.2. The meaning is demonstrated in Figure 13. If the effective height of the wall equals its height ie simple lateral support, any brickwork above or below the openings adjacent to the column should be ignored as its restraining effect is small. If enhanced lateral support is provided to the wall some measure of restraint is afforded by the adjacent brickwork. The effective height is then determined by interpolation between 0.75 and 1.0 $\times$ full height, by reference to the height of the taller opening.

Clause 28.3.1.4 enables the slenderness of a pier to be determined by treating it as a column or as part of the wall. The conditions are illustrated in Figure 14. The importance of this consideration of effective dimensions arises when additional load is carried by a pier by virtue of its reduced slenderness relative to that of the wall on its own. The pier is effectively stocky in the plane of the wall. If its overall thickness is limited to 1.5 times the wall thickness, the effective height normal to the wall may be taken as that appropriate to a wall rather than a column.

28.3.2 Effective length

Under some circumstances failure of a wall under vertical load may not follow the simple Euler buckling mechanism. When the length of the wall is much shorter than the height and when the vertical edges are restrained 'multiple buckling' occurs as if the wall is composed of a number of approximately square panels each of height equal to the length of the wall. The effective strength is much greater due to the restraint and the enforced mode of failure. The dimension controlling buckling becomes effectively the length of the wall, or some function thereof. Determination of the relevant length is related to the type of vertical lateral restraint as described in 28.2.3 and is illustrated in Figure 15. The assumptions behind this approach are based more on analogy with the buckling behaviour of thin steel sheets than on experimental tests on brickwork. However, it is

Figure 12 Effective heights for typical sections

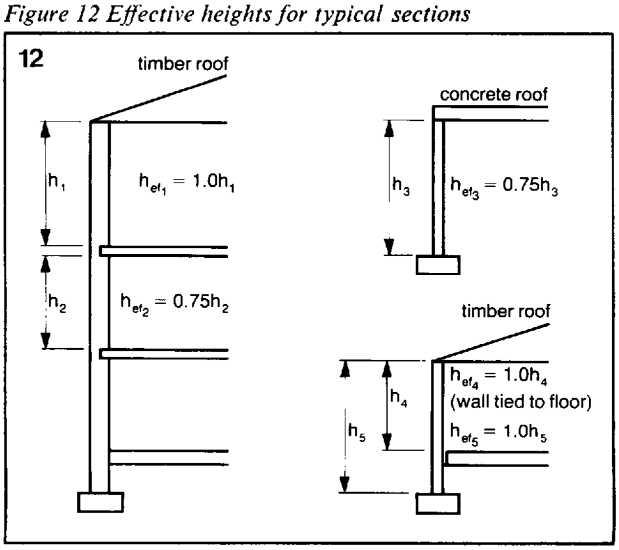


Figure 13 Effective height of a column between openings.

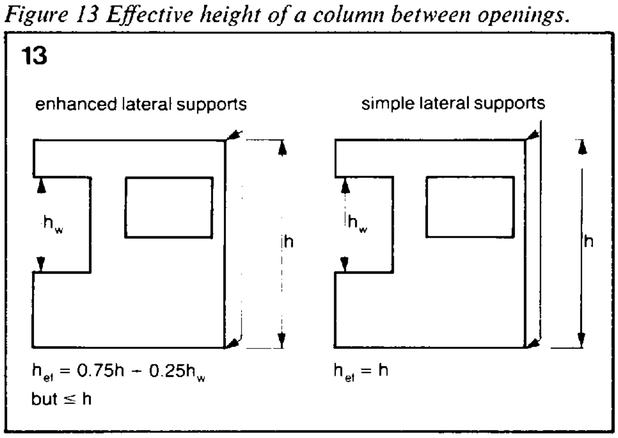
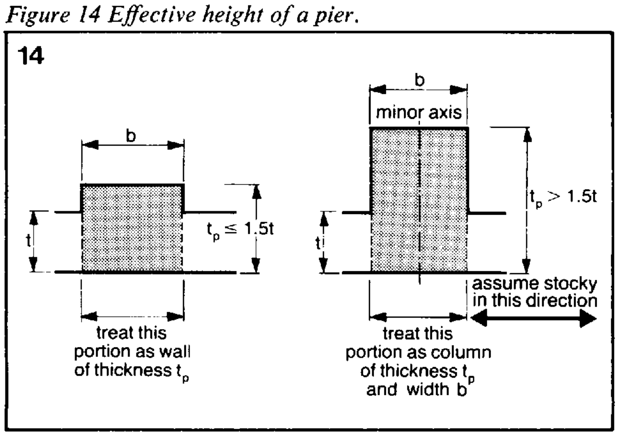


Figure 14 Effective height of a pier.



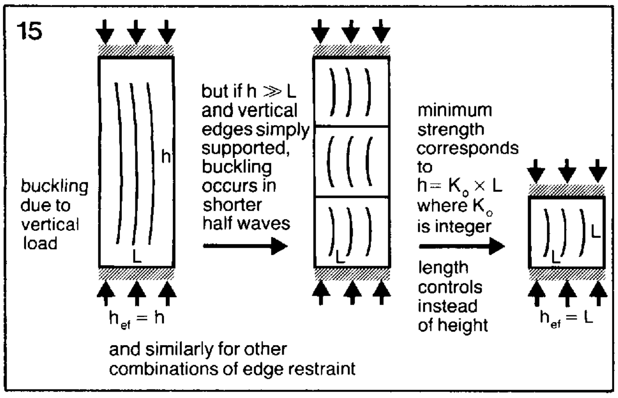


Figure 15 Effective length of a wall.

relatively rare for advantage to be taken of effective length in calculating slenderness. A typical example might be a lift shaft or stairwell in which all four walls buttress each other, or a short return wall supporting a substantially loaded lintel. An alternative approach is adopted in some countries where a continuous function is used to define a factor modifying effective height depending on the shape of the wall, and the degree of vertical edge restraint. The allowances appear generous and, based on current research in the UK, it is not clear that they are justified.

28.4 Effective thickness

For structural analysis in other materials, slenderness ratio is commonly defined as the ratio of effective length to radius of gyration. However, for solid walls the actual thickness, which is equal to the radius of gyration multiplied by $\sqrt{12}$, was introduced by CP 111: 1948 as the 'effective thickness' for calculating slenderness ratio. For timber, in fact, CP 112 provides alternative values for slenderness ratio based on both radius of gyration and effective thickness. The actual thickness could be modified to account for other arrangements in plan (piers, intersecting and cavity walls) which could stiffen a wall and so increase its effective thickness. The concept of radius of gyration was reintroduced to a limited extent in 1964 to cope with 'zig-zag' walls, which were irregular on plan but still essentially solid walls. However, these recommendations have not been included in BS 5628 for several reasons. Firstly, zig-zag walls, as such, are rarely used nowadays; secondly there are more advantageous ways of stiffening walls, eg, diaphragm or cellular walls which contain voids and are therefore no longer 'solid'; and thirdly, although the radius of gyration can be computed readily for such sections, the failure mechanism is no longer clear. The effect of slenderness on load carrying capacity therefore needs further examination. If such walls are used, it is recommended that the effective thickness be taken as the overall thickness of the wall, in the absence of experimental evidence suggesting greater freedom.

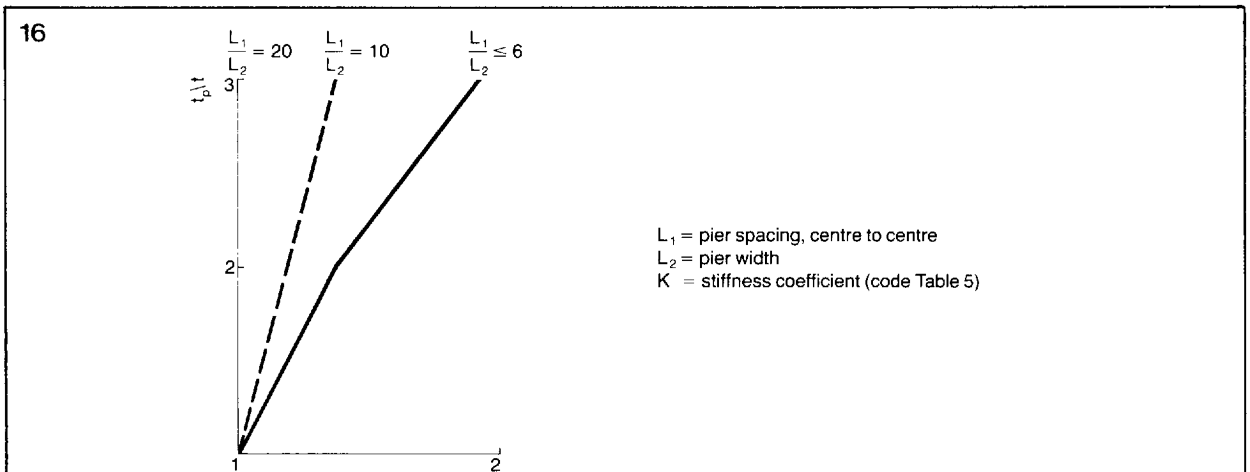
28.4.1 Walls and columns not stiffened by piers or intersecting walls

The origin of the rule given for the effective thickness of cavity walls is obscure, but is very nearly equivalent to adding the moments of inertia of the two leaves. That is, the two leaves stiffen each other but do not act compositely. When the thicker leaf exceeds twice the thickness of the inner leaf (which it might have done when cavity or 'hollow' walls were first devised) the effective thickness so calculated becomes less than that of the thicker leaf and should be ignored if the thicker leaf is the load-bearing leaf (also relevant to 29 and 32.2.3).

28.4.2 Walls stiffened by piers or intersecting walls

The data in code Table 5, based on limited work at BRS, have remained unchanged since the original publication of CP 111. The effective thickness is the thickness of an equivalent wall having the same moment of inertia as the wall with piers. The effect of changing the pier spacing and thickness is shown visually in Figure 16. The unity values in code Table 5 define the limits of application of the stiffness coefficient K. Whereas the effect of vertical lateral supports (28.2.3) is to alter the mode of failure of the wall (28.3.2), the effect of this limited degree of stiffening is confined to an effective reduction of slenderness, but still with the same effective height. In the case

Figure 16 Stiffening due to piers.



of bonded intersecting walls, it could be more advantageous to consider them as vertical lateral supports than as piers.

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Proc Inst Civ Engrs Part 2 v 65 pp 893–898 1978.

Fisher K and Haseltine B A.
Compressive loading tests on diaphragm walls.
5th Int Masonry Conf. Washington USA 1979.

29. Special types of wall

Just as the message in clause 20 was to consider the overall behaviour of the structure as a function of its component parts, so the message here, for composite walls, is to consider the interaction between the various materials and their effect on the behaviour of the wall as a whole. If a number of materials are designed to share the applied load, their interaction should be assured by appropriate attention to detail. Conversely, elements not designed to accept particular types of load should be isolated effectively from those that are. Although most of these recommendations are practical details they are often of vital importance to the structural performance.

29.1 Cavity walls

29.1.1 General

Various considerations lead to a determination of the slenderness ratio of a wall, taking into account its plan form and the type of edge restraint. However, its strength arises basically from the stresses which can be resisted by the cross-section under load. In a cavity wall, the cross-section is either that of a single leaf, or both leaves if both are loaded vertically (clause 32.2.3): it cannot be the gross plan area of the wall including the cavity.

29.1.2 Minimum thickness of leaves

This minimum is based on a consideration of the accuracy which is achievable for the plumb of a single leaf.

29.1.3 Width of cavity

The upper limit is a structural limit based on the ability of ties to be stiff enough to transmit forces due to buckling or wind load. Wider cavities might be acceptable on the basis of suitable wall-tests. The lower limit is more a serviceability limit at which cleaning out the cavity becomes difficult. Mortar droppings are likely to bridge the cavity and increase the passage of water to the inner leaf.

29.1.4 Spacing of ties

The spacing of ties is the same as given in CP 121 and represents traditional practice over the last 60 years or so. Although not derived by calculation, the density of ties over the face area of a wall seems intuitively right, particularly when extra ties are placed around openings and at angles or junctions with buttressing walls.

Tests have shown that, at the recommended spacings, the weakest tie specified by BS 1243 is adequate to transmit the buckling resistance required in a vertically loaded wall. Other tests have suggested that even fewer ties may serve this function. When lateral loads are to be shared between two leaves, the strength required of the ties should be assessed (clause 36.4.5), although in many cases wire ties may be found to be adequate. However, the experimental data on wider cavities is small and particular attention should be given to workmanship and supervision to ensure that the correct ties are installed at the correct spacings.

29.1.5 Embedment of ties

It is important that the minimum embedment is achieved with the ties at right angles to the cavity to avoid significant reduction in either pullout strength or buckling strength.

29.2 External cavity walls

The differential vertical movements between two leaves may be as much as 1 mm per metre height of wall, due to temperature differences and to differing moisture regimes and responses. Since most of these changes are reversible and repetitive, the adverse effect on the anchorage strength of ties could be considerable. The effects are cumulative with increasing height of wall so that, at three storeys, the differential movement at the top could be theoretically 8 mm which would impose severe strains on the bedding of ties. The relaxation to include four storeys is really to accommodate housing where experience suggests, particularly in Scotland where four storey dwellings are relatively common in cities, that the higher limit does not cause problems.

In higher buildings, masonry walling will often be built directly off a reinforced concrete slab. If the edge of the slab is not exposed but, for example, concealed by brick slips, particular attention should be given to achieving the accuracies of construction and preparation of surfaces necessary to achieve durable fixing of the slips. If the outer leaf of brickwork is built off a steel angle or similar support fixed to the floor slab, similar considerations apply. In addition, the corrosion resistance of the support and its fixings should be assured. The objective in limiting the height of masonry to 9 m is also to permit unrestrained movement vertically and care is necessary to provide adequate horizontal movement joints in the design. It is essential to ensure that they are achieved in practice and not, for example, filled with mortar or other debris. Although again a matter of detailing, and covered by CP 121, it is important to consider the need for vertical movement joints to accommodate horizontal movements.

29.3 Faced walls
The essential feature of a faced wall is that common action between the two compatible structural units is achieved by bonding, as in a solid wall composed of facing and common bricks. If the connection is provided by ties, mortar or grout the wall should be designed as a cavity, double-leaf or grouted cavity wall.

29.4 Veneered walls
Even though no structural action may be attributed to a veneer, adequate provision must be made for stabilising and securing the veneer to its supporting wall.

29.5 Double-leaf (collar jointed) walls

29.6 Grouted cavity walls
The principal reason for using these forms of wall is to achieve a fair-faced finish to both sides, of a standard that could not be obtained normally from a single leaf wall due to unavoidable variations in the dimensions of bricks and blocks. In the absence of the cross-bonding inherent in a single leaf wall wider than a single unit, strict control of constructional detail is necessary to justify consideration structurally as a single leaf wall. If the appropriate conditions are not met, design as the more conservative cavity wall is permitted.

The conditions are based on a limited number of tests in which it was shown that decreasing the stiffness and number of ties decreased the strength of the wall, and increasing the cavity width precipitated failure of the bond between the filling of the cavity and the bricks. The reference to flat metal ties arises because vertical twist ties will not sit in the bed joint properly with such a narrow cavity as used in collar-jointed walls.

The limitations on separation of the leaves for both types of wall arise primarily from the characteristics of the infill material and the practical aspects of placing it. The mortar is packed in as each layer of units is laid and derives its strength from its limited thickness. Because of the difficulty of achieving this with units more than about 75 mm high, the reduction is made to the quoted strengths for blockwork. As grout is essentially poured into the cavity, and may be placed and tamped in lifts of the order of 0.5 m, a minimum width of cavity is required.

In the experimental programme, the load was applied concentrically to both leaves, load being carried by the mortar or grout only by transfer through its interface with the bricks. The limitation on eccentricity was not tested experimentally and therefore should be treated with respect.

Beard R,
The compressive strength of some grouted cavity walls.
Proc Brit Ceram Soc April 1973 pp 113–140.

30. Eccentricity in the plane of the wall

31. Eccentricity at right angles to the wall

32. Walls and columns subjected to vertical loading
In order to determine the compressive strength of a wall required to resist a specific load, or to determine the vertical load bearing capacity of a given wall, it is necessary to determine the distribution of load on the wall, that is the effective eccentricity of the resultant vertical force. Distinction is drawn between variation of load intensity along the wall ie in its plane (clause 30) and across the wall or column (clause 31).

Along the length of the wall, the distribution will often be uniform unless the wall acts as a shear wall to resist, for example, wind forces. In that case, the load combinations (b) or (c) in clause 22 are applicable i.e. $1.4G_k + 1.4W_k$ or $1.2 (G_k + W_k + Q_k)$ unless the uplift combination $0.9\ G_k + 1.4\ W_k$ is more onerous.

Exceptional distributions of vertical load may also occur (see clauses 34 and 35). The horizontal forces may be transmitted directly to a single wall, eg, a spine wall, in which case the vertical forces induced in the wall may be calculated from consideration of simple equilibrium, as long as adequate connections are provided either along the vertical edge of the shear wall or by the floors along the horizontal edges. When the horizontal forces are resisted by several walls, the sharing of the load must be deduced from the relative flexibilities of the floors and walls. If the floors can be considered as rigid diaphragms, it will be conservative to distribute the forces between the shear walls in proportion to their stiffnesses on the assumption that they all deflect by the same amount and that adequate connections are provided. If the layout of walls on plan is asymmetrical, the possibility of torsion of the building about a vertical axis and further redistribution of forces may need to be considered. If the floors are relatively flexible when considered as horizontal diaphragms, eg partially interconnected precast units, the distribution of forces between walls will be deduced for each wall as if in isolation.

The desirability of calculating the effective height of walls from a knowledge of the relative stiffness of walls, columns and floors was discussed in clause 28. Such analysis will take into account the effects of vertical and lateral loads and continuity at junctions between walls and floors. The resulting moments may be expressed in terms of vertical load and effective eccentricities. However, it will not always be possible or appropriate to follow this procedure as the assumptions made may not be in keeping with the sophistication of the analysis. As an alternative approach, simplifying recommendations are made for determining eccentricities in clause 31. They are described in Figure 17. They represent approximations for common types of support or connection as envisaged in clause 28.2.2 (horizontal lateral support).

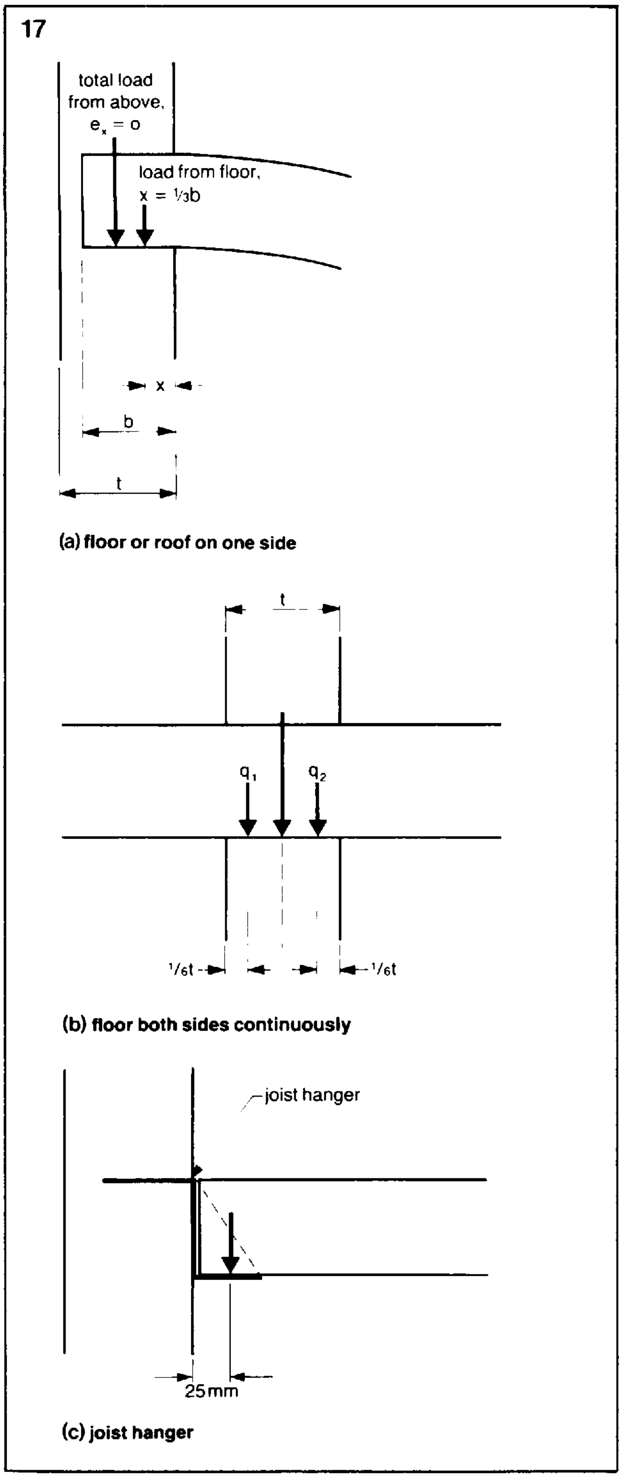


Figure 17 Assumed floor eccentricities.

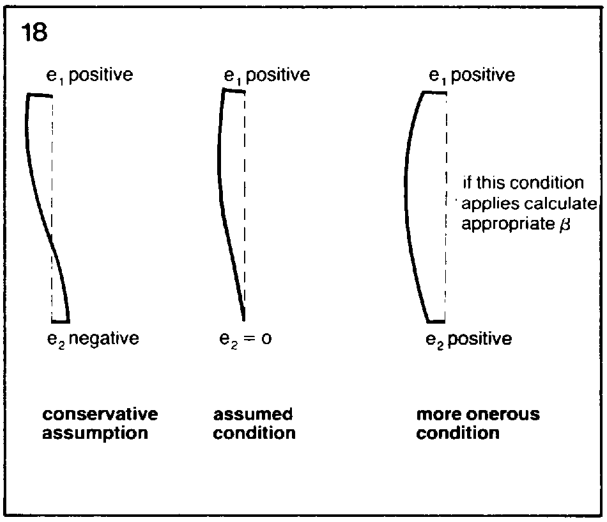


Figure 18 Combinations of resultant eccentricities.

In general, eccentricities might be expected at both top and bottom of a wall (see Figure 18). That at the top is due to the combined effects of load from the structure above, from the effects of horizontal forces, and from floor loads applied directly to the top of the wall. The eccentricity at the bottom is determined by the total load from above, and the degree of restraint afforded by the lateral support at that level. Following the argument shown in Figure 19 based on that used in CP 110 for a braced structure, code Appendix B assumes that the eccentricity at the bottom will tend towards zero ie, the vertical load at that level, immediately above a lateral support, may be assumed to be axial. This assumption is not intended to be conservative but a realistic assumption of actual conditions. A wall bent in double curvature due to top and bottom eccentricities of opposite sense could carry more load, but a wall with both eccentricities in the same direction represents a more onerous condition. If such conditions were seen to prevail then the recommendations in clause 32 would not be applicable.

In all cases of vertical loading (direct or induced by lateral loads), it is necessary to determine the effects of these eccentricities on the ultimate strength of the most critical sections of a wall. The method of doing so is described in clause 32 based on the analysis given in Appendix B, although it may be said at once that other approaches could be acceptable. In CP 111, the effects of slenderness and eccentricity have been allowed for by a series of stress reduction factors applied to the nominal basic stress for the cross-section under consideration. These factors have been based on a considerable volume of experimental data obtained by BRS, initially on columns or piers, and then on walls, and more recently at BCRA, and also in the USA. As experience has been gained in using the factors, and as the experimental base has increased, the reduction factors have become less stringent.

In reviewing this aspect of performance in the light of the limit state approach, an analytical basis for determining the effects of slenderness and eccentricity seemed appropriate. This basis is not a necessary consequence of limit state approach but has been verified, of course, against experimental data. The approach used by CP 110 for plain concrete walls has been adopted. It is assumed that at ultimate failure the stress distribution in a wall at the critical section may be represented by a rectangular stress block and that, when significant, the effects of additional moments induced by lateral deflection of the wall under vertical loads are taken into account. Finally, the assumption that only braced vertical load-bearing masonry walls should be built, or at least that the recommendations in the code apply only to such walls, leads as described above to the assumption of zero eccentricity at the bottom of a wall. The models representing these conditions are shown in Figure 20 (stress block) and Figure 21 (deflection of wall).

Based on the deflections in the central region of the wall, the additional eccentricity due to the effects of slenderness ratios greater than 6 is given

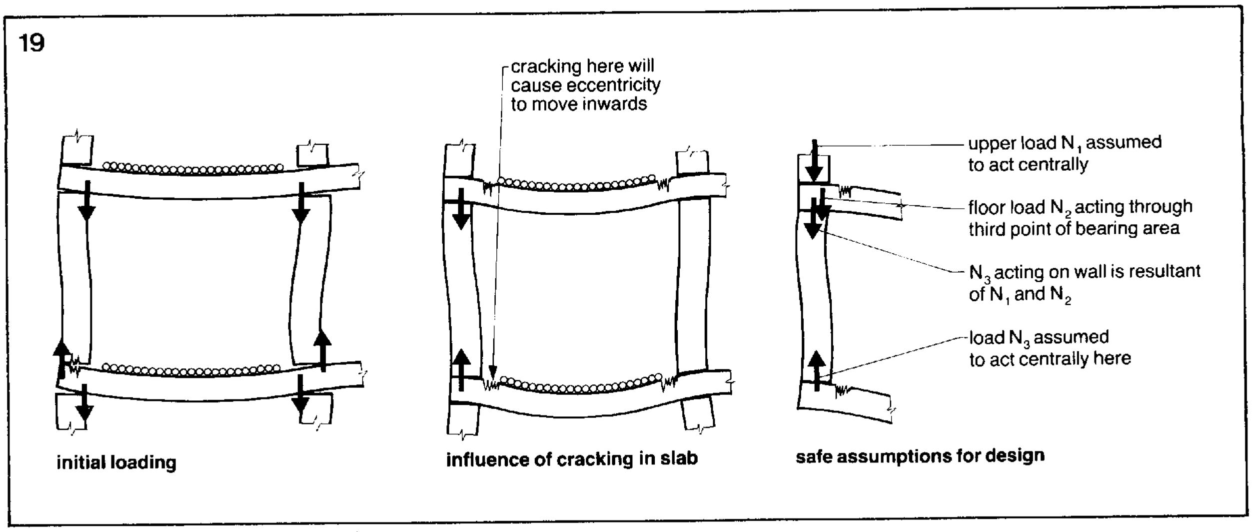


Figure 19 Eccentricities in a braced structure.

by $e_a = t\left[\frac{1}{2400}\left(\frac{h_{ef}}{t_{ef}}\right)^2 - 0.015\right]$ (Appendix B, equation 1). This function is plotted in Figure 22 to indicate its magnitude in relation to the eccentricity at the top, e_x. The allowance is marginally more onerous than that given in CP 110, that is, a slightly larger deflection is assumed. However, for stocky walls and columns, the additional effect is ignored. From Figure 21 it is seen that the largest eccentricity occurs in the mid-height region and, generally the total eccentricity, $e_t = 0.6e_x + e_a$ (Appendix B, equation 2) because e_x varies linearly from top to bottom of the wall. However, when e_a is small the total calculated eccentricity at the top of a wall, e_x, may be larger than e_t; design should be based then on this larger value, known as the '*design eccentricity*', e_m.

By relating the width and magnitude of the stress block to the design eccentricity (which includes the effects of structural and additional eccentricity), and to the characteristic compressive strength a '*capacity reduction factor*', β, may be deduced to yield directly the load which a wall can carry. In CP 111 the use of a stress reduction factor of course requires a further calculation, based on an assumed stress distribution, to derive the load. The magnitude of the ultimate compressive stress is f_k, or as a design stress $\frac{f_k}{\gamma_m}$.

When the design eccentricity ie the larger of e_x and e_t, is small, that is less than 0.05t its effects are negligible and the full section of the wall resists failure stresses; there is no reduction in capacity required and $\beta = 1$. For larger values of e_m, the width of the stress block is $t\left(1 - \frac{2e_m}{t}\right)$.

It is important to distinguish clearly between e_m (a function of e_x) which is used to calculate the effects of eccentricity and slenderness, and e_x which is used in code Table 7 as a design parameter. It should be noted also that h_{ef} in code Table 7 encompasses the lesser of effective length or effective height.

Logically, the magnitude of the stress block at failure should also equal $\frac{f_k}{\gamma_m}$ even when e_m is greater than 0.05t. For convenience in design, however, it has been assumed that no further calculation should be necessary for applied eccentricities e_x less than 0.05t. If a stress block of

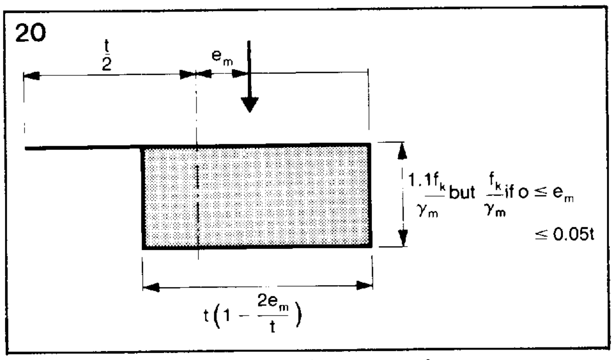


Figure 20 Stress block under ultimate conditions.

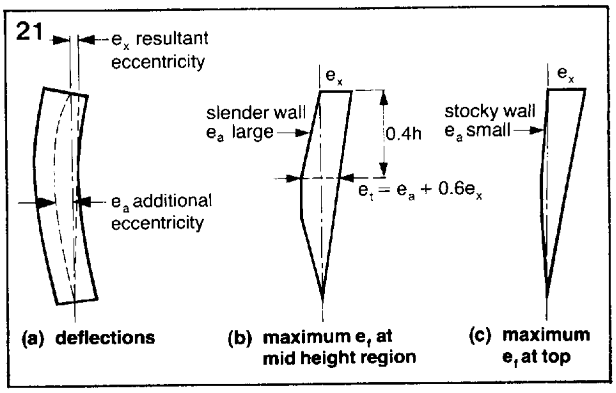


Figure 21 Design eccentricity.

$\frac{f_k}{\gamma_m}$ were retained there would be an unreasonable step reduction in β as e_x exceeded 0.05t. By increasing the stress by 10% a smooth change is provided leading to a value of $\beta = 1.1\left(1 - \frac{2e_m}{t}\right)$. (Appendix B, equation 4). Values of β calculated from this expression are given in code Table 7.

A comparison between assuming a rectangular stress block and a triangular stress distribution, as suggested in CP 111, is shown in Figure 23. It also illustrates the effect of not requiring calculation for $e_x < 0.05t$.

The key to the acceptability of this whole approach lies in correlation with the experimental evidence. As the values of β stand, they represent in many cases a considerable increase in allowable load capacity compared with CP 111. A calibration exercise was carried out by Jenkins & Potter on behalf of the Building Research Establishment, with contributions from the Cement and Concrete Association and the Property Services Agency of the Department of the Environment. The aim of

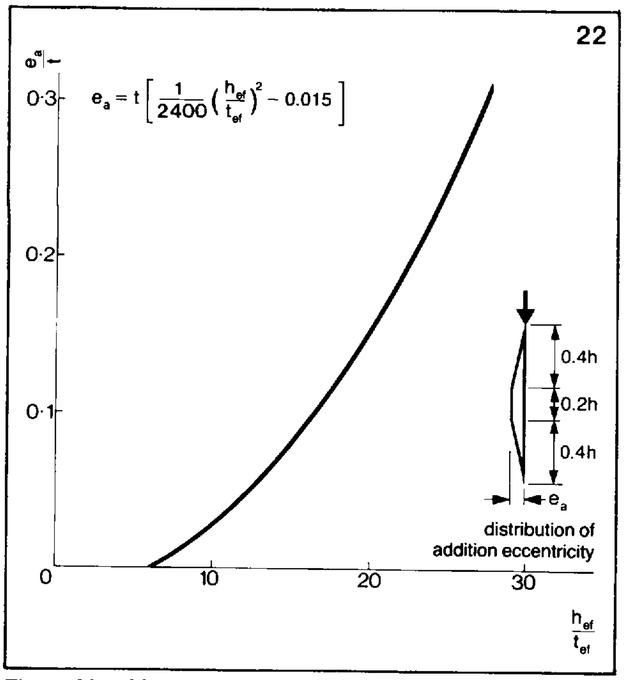


Figure 22 Additional eccentricity v. *slenderness ratio.*

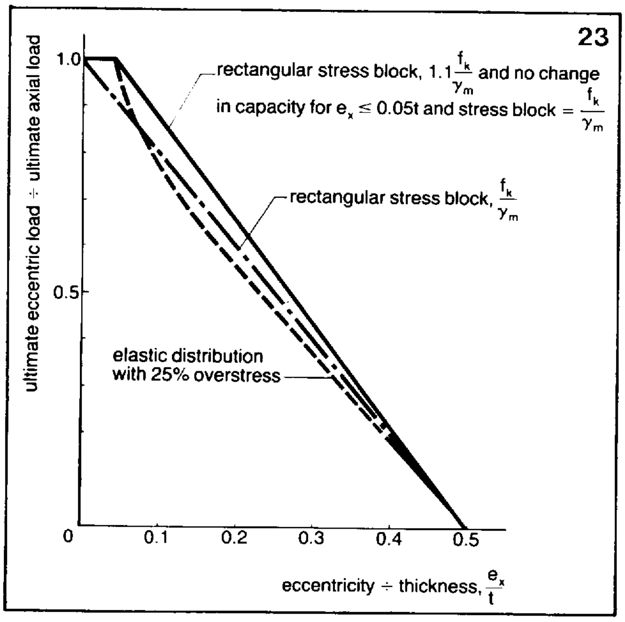


Figure 23 Comparison of stress blocks with no slenderness effect.

the exercise was to check that the proposed new code would not result in designs for the use of structural masonry which were significantly less safe than, or otherwise incompatible with, current practice. A major part of the calculations was concerned with the vertical load capacity of walls and columns of various typical thicknesses and shapes. For each type of construction nine combinations of brick, block and mortar strength were considered under four combinations of dead and live load with five slenderness ratios and four values of eccentricity. As a result, a number of alterations were made to the draft code and recalculation carried out where necessary.

An overall comparison is afforded by the histogram in Figure 24 where the individual calculated results are plotted on the basis of the ratio of the capacity allowed by CP 111 to that allowed by BS 5628. From an examination of the individual results which make up this composite picture, it is clear that for axial loads the two

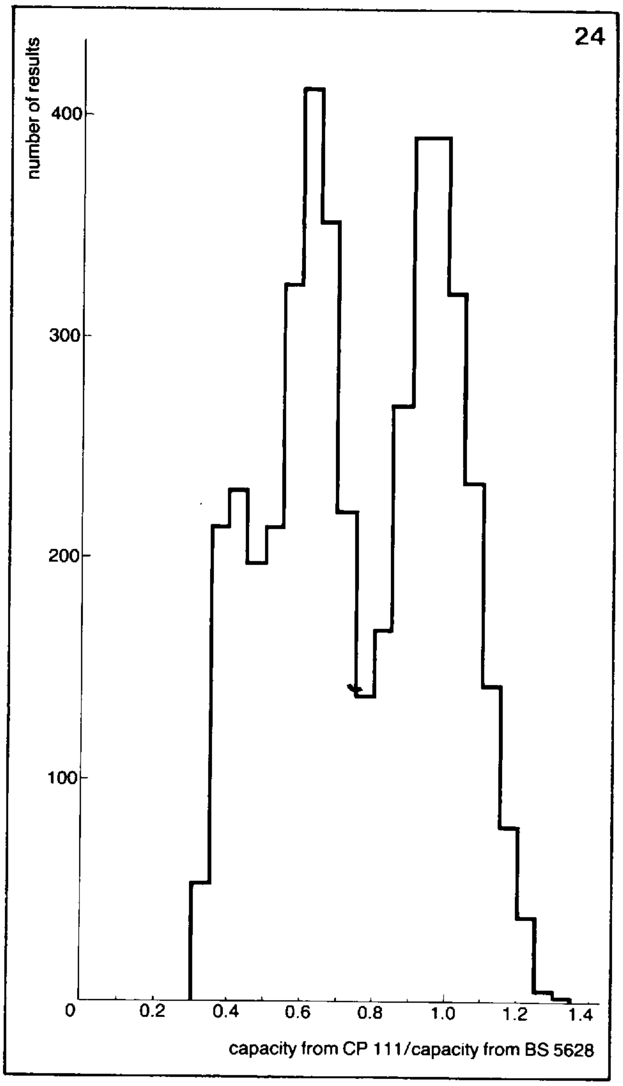


Figure 24 Comparison of compressive load capacities.

codes give broadly similar results. The divergence occurs as eccentricity and slenderness increase. A direct comparison with CP 111 was made difficult by the different partial safety factors ascribed to dead and live load. As the ratio of live to dead load is increased from zero to three BS 5628 becomes about 10% more onerous than CP 111, all other conditions being the same. Primarily as a result of this there are circumstances for most bricks and blocks which are slightly more onerous than before. Calculations were also performed for the walls of a specific design of a two-storey house, an eight-storey cellular plan block and a nine-storey cross wall block. These results showed that, when using $\gamma_m = 3.1$ for bricks, BS 5628 allowed bricks to be used of up to two classes lower strength than when designed using CP 111.

However, the major reason for the differences at eccentric loads lies in the assumption of zero eccentricity at the bottom of a wall. The stress reduction factors have represented hitherto lower bounds to a mass of experimental results which for the most part have been derived from tests in which the eccentricities were applied at both top and bottom. In other words, they did not represent conditions comparable to those assumed now. By isolating and concentrating attention on the experimental data which do represent the assumed loading conditions Figure 25 demonstrates that the less conservative reduction factors are justified. A small part of this data is derived from tests at BRS but much of it relates to tests in the USA on both bricks and blocks.

Cavity walls

It must be made absolutely clear that the thickness of a wall or column referred to in 32.2.1 and 32.2.2 is the thickness of a single leaf wall or of the loaded leaf of a cavity wall. Although 't' is defined in clause 4 as the overall thickness this does not apply in the context of a cavity wall and is not used in code Figure 2. The guidance given in clause 32.2.3 applies only to the case when both leaves support vertical load and the resultant is contained between the centroids of the two leaves. When the resultant acts outside this central region the load may be assumed to be carried by one leaf only.

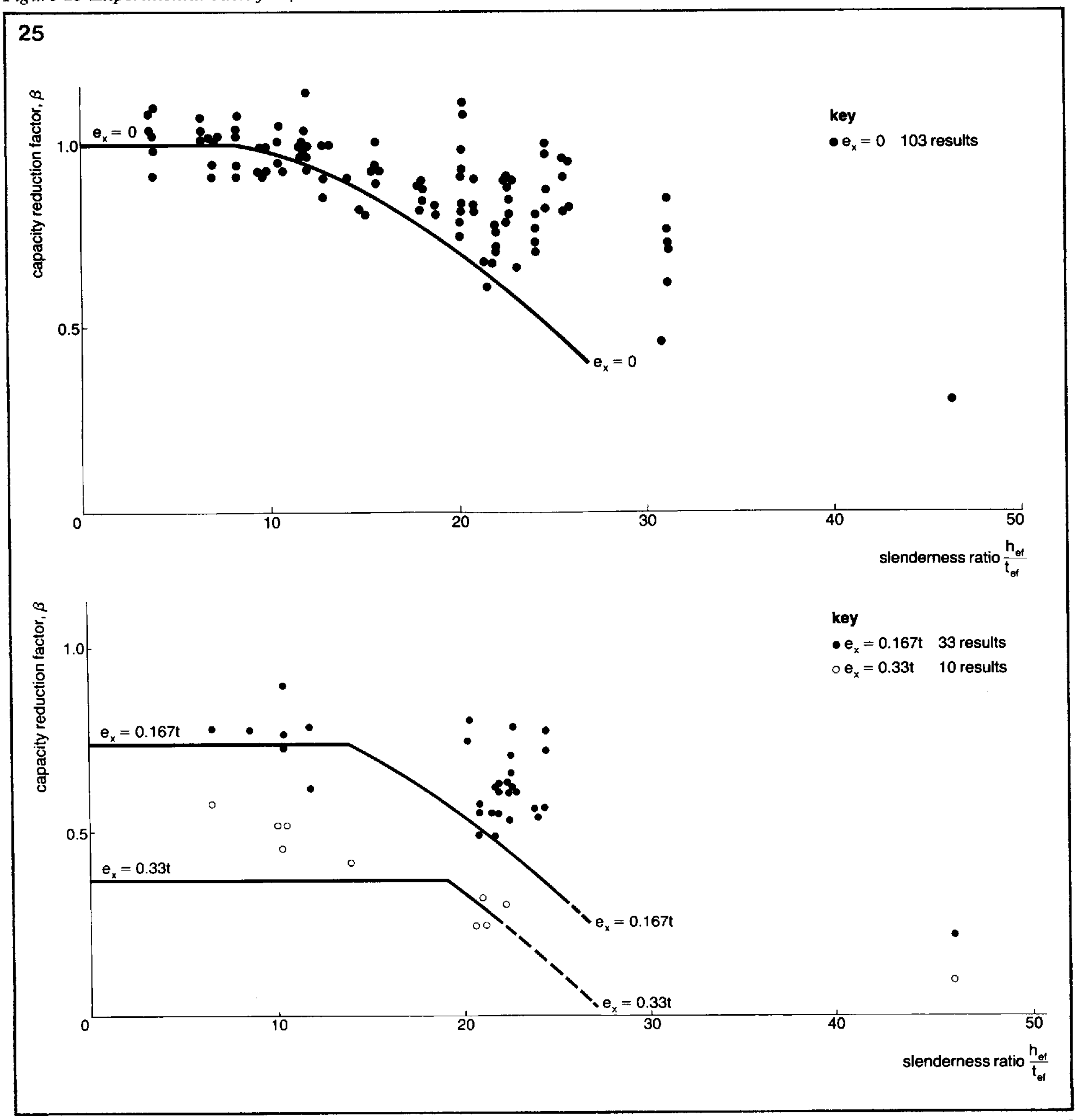
Since there are several clauses at various junctures in section 4 which refer to different aspects of cavity walls it is convenient and desirable to list them together here, excluding those relating only to lateral or accidental loads.

Lateral supports:	Horizontal 28.2.2	Vertical 28.2.3
Effective height of piers:		28.3.1.4
Effective thickness:	28.4.1 (two-thirds rule) 28.4.2 (stiffening by piers or walls)	
Loaded area:	one leaf 29.1.1	both leaves 32.2.3
Limitations on plan:	thickness 29.1.2	cavity width 29.1.3 embedment 29.1.5
Limitations on face:	tie spacing 29.1.4	height 29.2

33. Walls subjected to shear forces

The recommendations suggest that the design shear stress should be calculated on the assumption of a uniform distribution over the horizontal cross-section of the wall resisting the horizontal load. If it should be more appropriate

Figure 25 Experimental basis for β.



to assume an alternative distribution, eg parabolic, it will be necessary to calculate the maximum shear stress and check that against the design shear strength. The design shear stress will involve the appropriate partial safety factor from clause 22. It is assumed that the walls have a simple rectangular plan and that no account is taken of intersecting walls.

Stafford Smith B and Rahman K M K,
The variation of stresses in brickwork walls subject to shear forces.
Proc 3rd Int Brick Masonry Conf. Ed L Foertig and K Göbel Bonn Bundesverband der Deutschen Ziegelindustrie 1975.

34. Concentrated loads: stresses under and close to a bearing

The recommendations in CP 111 allow a 50% increase in permissible stress, due to concentrated loads. However, the meaning of 'local' is left to the designer's discretion. Although experience has not found this allowance generally wanting, there are theoretical and experimental grounds for restricting the increase in some cases, and indeed for relaxing it in others. The new recommendations seek to define these conditions so that a more realistic approach is adopted.

When a small central area of a pier is loaded, the capacity of the pier has been shown to increase several fold. The masonry beneath the loaded area develops a state of triaxial stress which enhances its ultimate strength compared with a uniformly loaded pier. Similar enhancement occurs when a load is applied to a short length of a wall, but other factors then become significant as well. The length to width ratio of the loaded area, the proportion of length and width of wall to which load is applied, and the proximity of the ends of the wall all need to be considered. The experimental data is not extensive and relates to full width bearings of various lengths and positions on slender walls, and to central bearings as various proportions of the cross-section of square columns.

Figure 26 (which reproduces Figure 4 of the code, with annotations) attempts to present a balanced assessment of the importance of the factors in terms of allowable increases in compressive strength in steps of 25%, 50% and 100%. Figure 27 sets out a logical connection between the various types of bearing on the basis of the effects of length to width ratio of bearing area and of the ratio of bearing width to wall width. The starting point is bearing 2.3 in Figure 26 in which more than half the width is loaded. A maximum allowable length of four times the width of the wall has been assessed on the basis of the rate of dissipation of vertical stress with depth beneath the bearing and the limited experimental data (unpublished). The allowable overstress of 50% corresponds to current practice. As a rough guide the stress reduces by 50% over a depth below the load equal to the length of the bearing. Whereas this implies significant reduction within a short distance below a short bearing, beneath a long bearing there will be a substantial zone of significantly increased stress within which failure of the wall would occur.

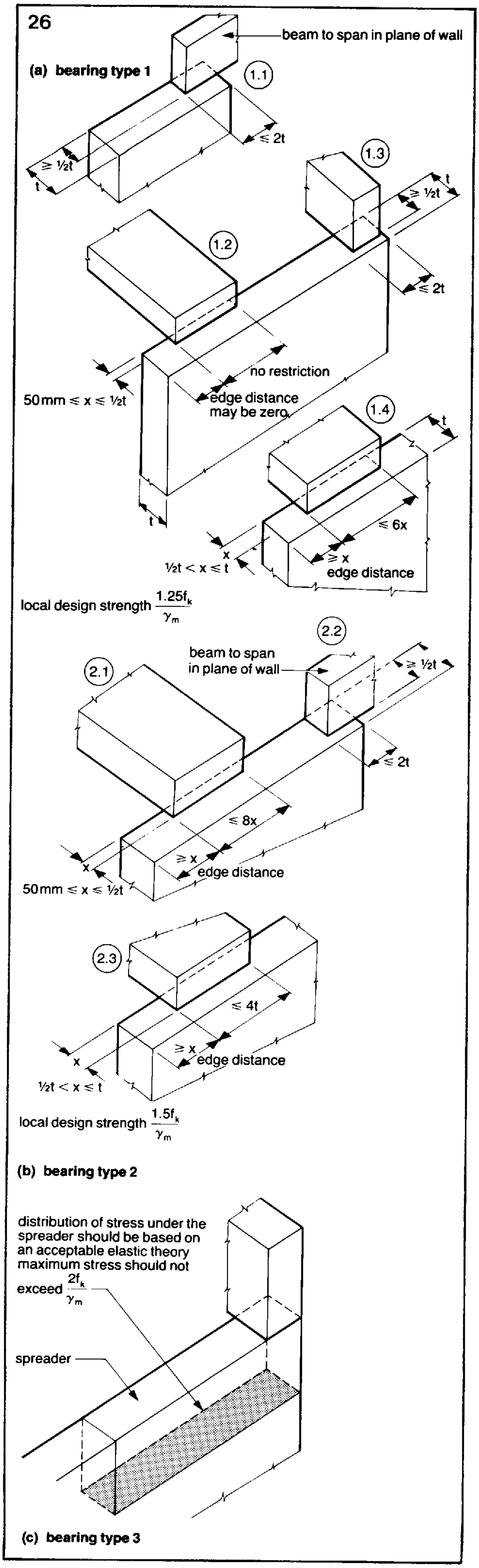


Figure 26 Bearings for concentrated loads.

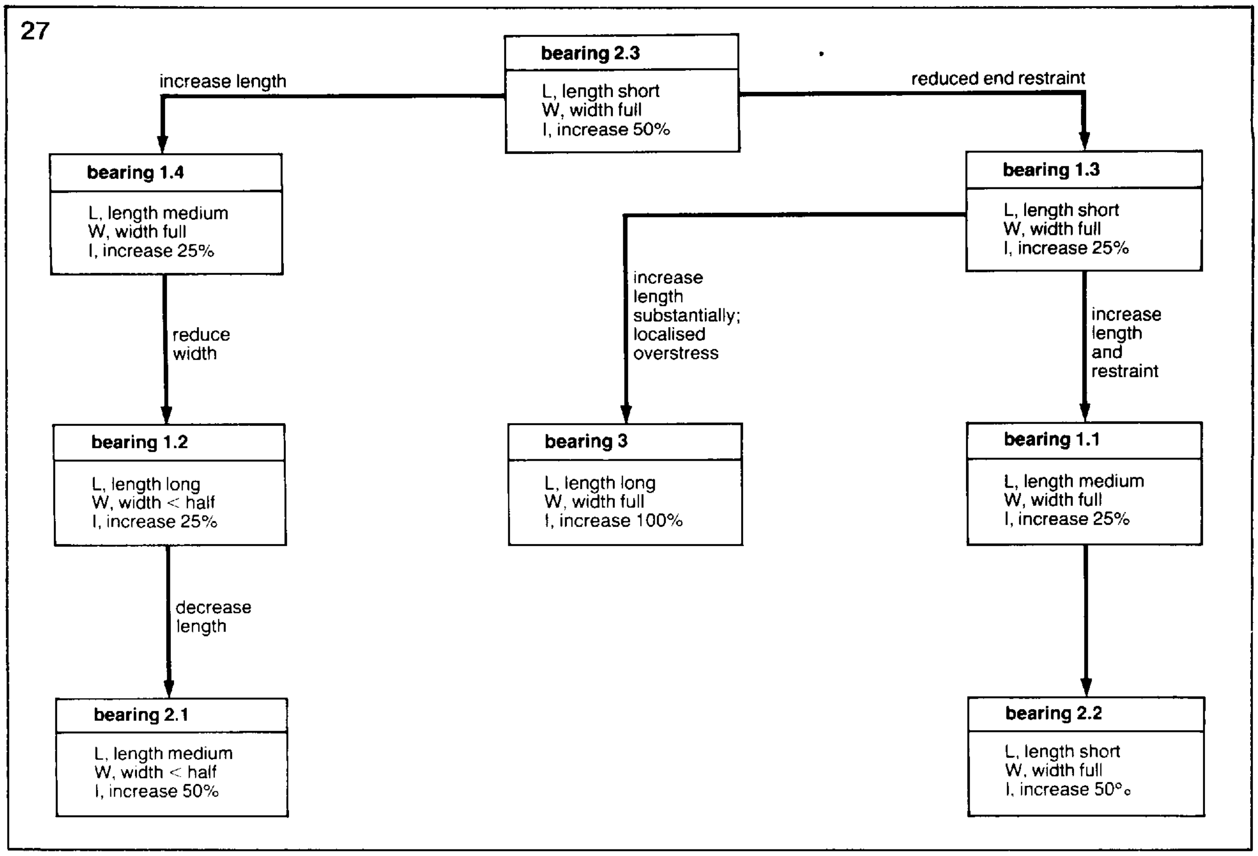


Figure 27 Relationships between bearing types and local increases in stress.

In order to allow an increase in bearing length the overstress must be limited. Bearing 1.4 in Figure 26 therefore allows an increase of only 25% of compressive strength which provides a possible increase in total load of 25% compared with the shorter higher stress bearing. A similar allowance is permitted for bearing 1.2 in which the bearing width is less than half the width of the wall, but greater than 50 mm, a reasonable minimum bearing. There has been no experimental work on this type of partial bearing on masonry but limited tests on concrete suggest that the 25% allowance is reasonable. Rupture occurs effectively by shear failure of the wedge of material beneath the bearing, modified by any restraint on movement of the loaded beam perpendicular to the wall. By restricting the length of this type of partial bearing, further restraint is gained from the ends of the failure zone and the 50% allowance of bearing 2.1 becomes feasible. The maximum length of this type of bearing is the same as that allowed for a full width bearing, 2.3.

Tests with loads at or near the ends of walls confirm that when the bearing length is limited (to for the sake of convenience, 2t) only a 25% overstress is justifiable (bearing 1.3). This arrangement is effectively 'half' type 2.3. When the failure mode is constrained by the restraint offered by a beam spanning in the plane of the wall, some enhancement is possible, either as an increase in bearing length to 3t (type 1.1), or as an increase in overstress to 50% (type 2.2, with maximum bearing length of 2t). In all these cases of end loading, the width of bearing should exceed half the width of the wall.

In other cases of concentrated loading at the ends of a wall, a spreader should be used as in bearing type 3 (code Figure 4c). Then the overstress may be 100% since it applies over only a very short strip of wall. Whereas it will be sufficient to assume a uniform stress beneath the concentrated load in most cases, the stress distribution beneath a spreader loaded at the end of a wall will be distinctly non-uniform and the value of the maximum stress should be considered. Although an elastic stress distribution is reasonably acceptable the presence of the vertical boundary means that numerical solutions will be the most practical in cases when standard solutions for rigid beams on elastic foundations are not appropriate. It is worth noting the general similarity between a spreader supporting a concentrated load at its end and the configuration obtained by inverting a composite wall-beam (see clause 35) supporting a uniform load and simply supported at each end. Each reaction corresponds to an applied concentrated load. However, since a spreader is of more restricted length than the supporting beam the stress concentration will be more severe. Although the typical distribution illustrated in code Figure 5(b) appears to be out of equilibrium with the applied load, this obviously cannot be and local yielding of the masonry and flexure of the spreader will occur. The illustrated distribution was obtained from an experiment on a spreader.

In most cases of spanning at right angles to the wall the distance of the bearing area from the end should exceed a minimum value as shown because the increase in stress derives in part from the restraint at each side of the bearing. In addition, although it is convenient to assume a uniform stress distribution on a bearing surface, there will tend to be stress concentrations beneath the edges of a relatively rigid beam or spreader. For a long bearing type 1.1 the contribution would be negligible and the condition does not apply.

The discussion so far has assumed that only concentrated loads are acting. In practice, there will often be additional distributed loads and in such cases the combined stresses should be checked against the appropriate allowable increases. Since the dissipation of stress, even at limited bearing lengths, occurs over a finite depth down the wall it is possible that there will be a significant increase of stress in the central zone of maximum eccentricity in a slender wall. The stresses should be checked at a depth of 0.4h using the appropriate capacity reduction factor and assuming that the stress disperses within planes at 45° to the horizontal as shown in Figure 5 in the code.

Attention has been drawn to minimum edge distances, but it should not be assumed that they can be applied on both sides of a bearing. This would lead to bearing areas as much as 75% of the cross-section of the wall for which no enhancement of strength can be justified. In fact, it is unwise for a bearing to exceed even 25% of the cross-section of a wall if a 50% enhancement is required, particularly if the wall is very short.

The calibration exercise referred to before also included calculation of the maximum concentrated load which could be carried on a 215mm brick wall and a 200 mm block wall (unit strengths 20.5 N/mm² and 7 N/mm² in mortar designation iii) by a central padstone 400 × 200mm wide and by a beam spanning at right angles to the wall. A range of loading conditions, eccentricities and slenderness ratios was considered. On the whole BS 5628 gives lower capacities than CP 111, more so at axial than at eccentric loads. The limited calculations suggest that usually the check at the level of the bearing will control, rather than at 0.4 h.

Bearing Stress Investigation: Report on Bearing Pressures on Brick Walls.
Anon. Structural Engineer August 1938 pp 242–268.

Hendry A W Bradshaw R E and Rutherford D J,
Tests on cavity walls and the effect of concentrated loads and joint thickness on the strength of brickwork.
CPTB Research Note v 1 n 2 1968.

35. Composite action between walls and their supporting beams.

The original work on the composite action of brick walls supported on reinforced concrete beams comprised experimental tests at BRS published in 1952. Since then there have been many analytical studies of the problem using a variety of techniques, some leading to methods suitable for design. More recently there has been further experimental work, including walls on steel beams.

Much of the interest has lain in studies of the bending moments, shear forces and tensile forces induced in the supporting beam. Generally, the beam may be designed to support loads much smaller than caused by a uniformly distributed load, because arching action within the masonry concentrates the vertical stresses near the ends of the beam (Figure 28). Depending on the relative

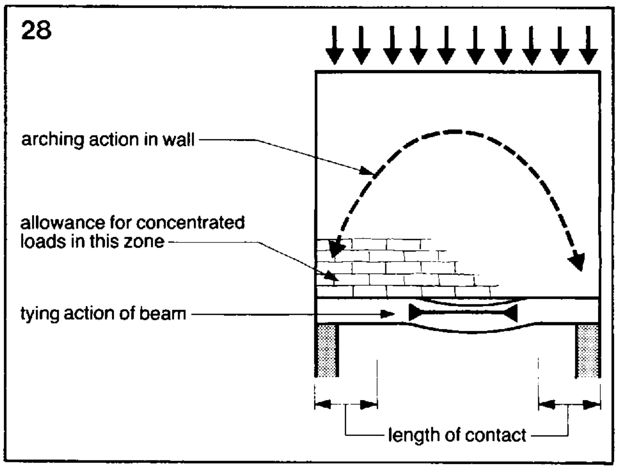


Figure 28 Composite action with in-plane arching.

stiffness of the wall and the beam, there will be a central zone in which the masonry is not in contact with the top of a flexible beam. A check on deflection of the beam may be necessary, and also on shear near the supports.

Three principal conditions determine the ability of the masonry to form an arch, although there are further conditions imposed on the supporting beam which are not of immediate concern here. Firstly, it has been shown that if the ratio of height to length of the wall is less than 0.6 there is insufficient depth within which an arch can develop. Secondly, openings must not be placed within the lines of thrust of the imaginary arch. This aspect has not been studied sufficiently to enable precise limitations to be set, but generally an annulus centred on the midpoint of the beam (length L) with radii 0.25L and 0.6L should be imperforate. Finally, the increased stresses imposed by the arching action must not exceed the local compressive capacity of the masonry.

Determination of the relative stiffnesses of beam and wall can lead to an assessment of the contact length over which the increased stresses may be assumed to act. By relating this bearing area to the increases in local stress suggested by clause 34 and code Figure 4, the average concentrated stress near the ends of the beam may be checked. The stress concentration spreads out relatively quickly upwards from the beam so that the local effects of applied and additional eccentricities may be ignored. Since the ultimate reserves of strength in a wall are used to some extent by the stress concentrations, the zones at the ends of the beams are likely to offer the critical conditions.

Composite action of this type was originally investigated for the design of brick walls on beams supported by piled foundations for low-rise housing. Subsequent work extended the applicability to more heavily loaded walls, both cases permitting the designer to achieve economies and more efficient design *ab initio*. However, it is common also for composite action to be invoked when assessing the strength of existing structures as a result, for example, of damage or possible changes of loading. Under these circumstances, it is particularly important to assess carefully the tying action of the beam. When the end of a supported wall occurs at a

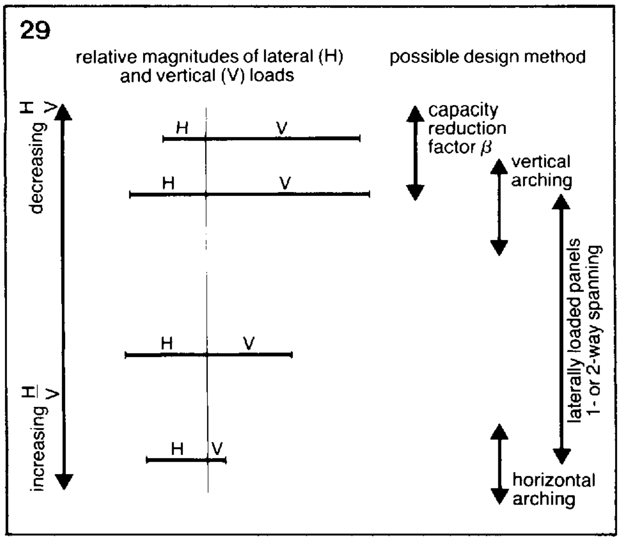


Figure 29 Relationships between design methods for laterally loaded walls.

corner of a building, there is no in-plane restraint to the arch other than afforded by the beam.

A further common circumstance investigated only recently is the effect of loads applied directly to the supporting beam, as for example by floor joists spanning on to the beam. A modification to the design procedure for a beam loaded only by a wall is possible. However, the stiffer beam required to resist the additional applied load increases the contact length with the wall and so reduces the benefits of composite action.

Wood R H,
Studies in composite construction. Part 1 - The composite action of brick panel walls supported on reinforced concrete beams.
National Building Studies, Research Paper No 13, London HMSO 1952.

Timoshenko S,
Strength of Materials: Part 2.
D van Nostrand Princeton 1957.

Wood R H and Simms L G,
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BRS CP 26/69.

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Studies in composite construction: an elastic analysis of wall-beam structures.
BRS CP 15/69.

Burhouse P,
Composite action between brick panel walls and their supporting beams.
BRS CP 2/70.

Riddington J R and Stafford Smith B,
Composite method of design for heavily loaded wall-beam structures.
Proc ICE Part 1 1978, 64 137–151.

Davies S R and Ahmed A E,
An approximate method for analysing composite wall/beams.
6th Int Symp on loadbearing brickwork, London 1977, Proc Brit Ceram Soc n27 Dec 1978.

36. Walls subjected to lateral load

Specific recommendations for designing walls to withstand lateral load are one of the major new features of this code. They result from a substantial volume of research designed to understand the mechanisms involved in masonry walls under the action of forces normal to their plane. The work stemmed both from a need to exploit the lateral strength of masonry to the limit to cope with accidental loading requirements and from uncertainty about the margins of safety existing in panel walls that were being built, particularly following the upward revision of the magnitude of extreme wind loads. Thus the recommendations relate to panel walls with only edge restraint as well as to walls in which vertical or horizontal in-plane forces are developed or applied.

The arrangement of sub-clauses follows broadly the same logical sequence adopted in clauses 28–32, that is: (1) supports and connections—36.2; (2) limiting conditions—36.3; and (3) design procedures:
(a) low in-plane forces—
36.4.2, 36.4.3 (panel walls)
(b) high in-plane horizontal forces—
36.4.4 (horizontal arching)
(c) high in-plane vertical forces—
36.8 (vertical arching)
(d) free-standing—
36.5
(e) earth pressure—
36.6, 36.7
The relationship between the various approaches is shown diagrammatically in Figure 29.

36.1 General

The opening reference to the empirical guidance given in CP 121: Part 1 relates to the amendment no. 1, 1975, but the recommendations therein should be treated circumspectly. The amendment gives limiting sizes for laterally loaded panels with relatively little restriction on their application. When applied to average materials in conditions of medium exposure the recommendations are broadly·acceptable. However, if used at the extreme conditions permitted by CP 121, the panel sizes are difficult to justify using the recommendations of BS 5628. When CP 121 is next revised, it is to be expected that the 'rule of thumb' guidance will be modified so as to give conservative limitations in relation to dimensions, calculated from BS 5628.

36.2 Supports and continuity

Because the degree of edge restraint (translational or rotational, as in code clause 28) can alter the resistance of a panel of a given size and type of masonry by nearly an order of magnitude, it is essential that the assumptions made in design reflect what can be achieved in practical construction. Although such a range of strengths represents an extreme case, an unjustified assumption of continuity on one edge of a panel in place of simple support could, in reality, lead to a panel being 30% weaker than expected. As a general rule, it may be assumed that simple support only (ie no rotational restraint) exists at all but a free edge, unless the masonry is bonded

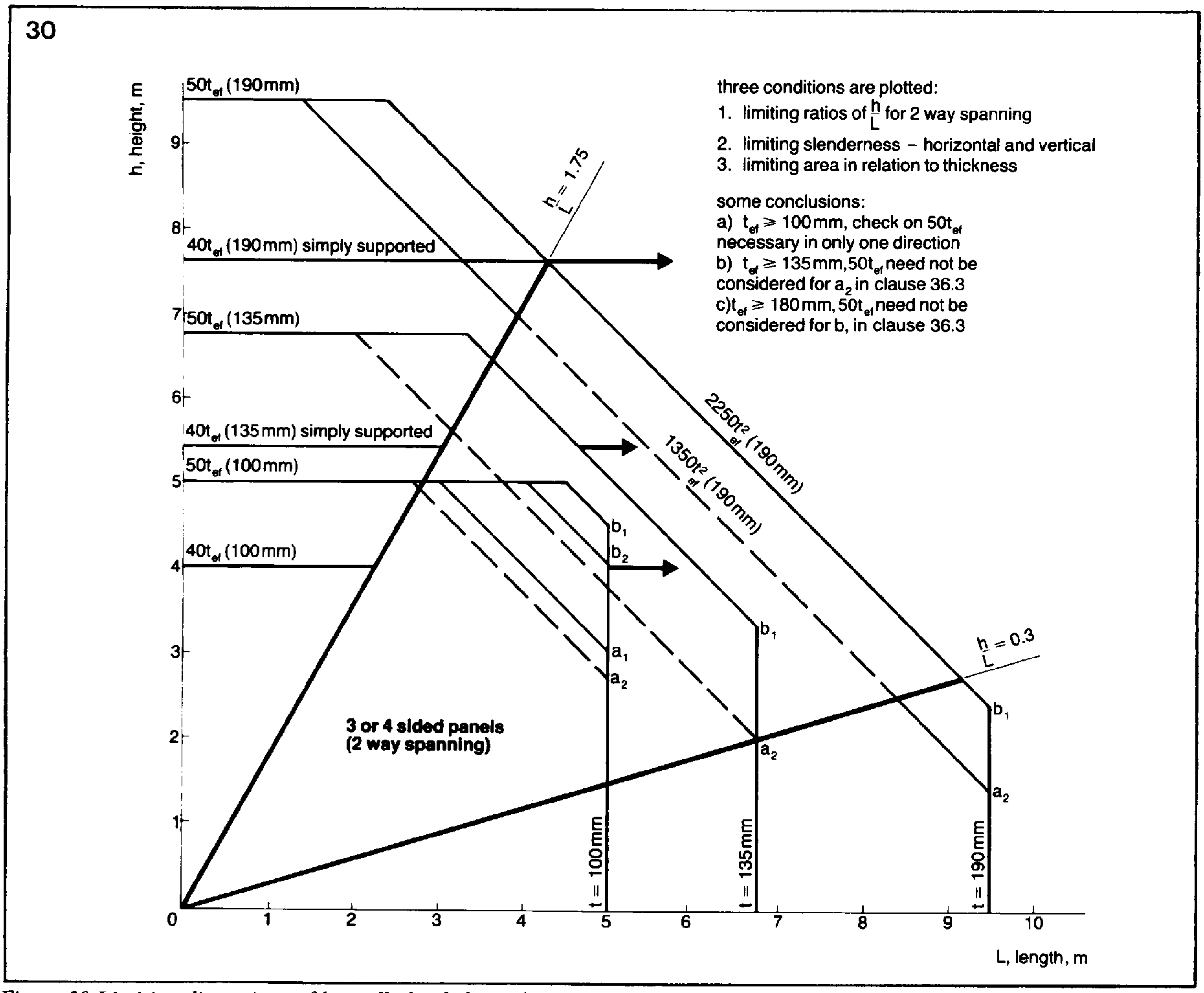


Figure 30 Limiting dimensions of laterally loaded panels.

into return walls or is in intimate and permanent contact with the superincumbent floor or roof. A good selection of practical conditions are given in code Figures 6 and 7. It may be possible to assess conditions in which less than full rotational restraint exists, but advantage is likely to be gained only for panels with a free top edge or one or both vertical edges free.

While the adequacy of rotational restraint will be assessed generally on the basis of the form of the construction, it is possible, and necessary, to check by calculation the translational restraint, that is the shear, tensile or compressive strength of the connections. The strengths of tied connections (code Table 8 and, for compression, in the text clause 36.2) are based on limited tests on a range of ties and anchors. Shear or frictional restraint may be checked using clauses 25, 26 and 33.

It is worth drawing particular attention to the last paragraph of clause 36.2. It is essential to tie together adequately the edges of both leaves of a cavity wall when, as is frequently the case, only one leaf is connected to the supporting structure. Knowledge of the strengths of ties is also needed when checking the transfer of load over the full face of the two leaves, as well as locally at supports. Although the strengths of ties in code Table 8 and clause 36.2 are stated in the context of connections, it is reasonable to assume that the relevant values apply also to cavity wall ties.

36.3 Limiting dimensions

It is essential to treat the dimensions provided by this clause as checks to be carried out independently of the strength design procedures. They are not design calculations in themselves, but are akin to the limiting slenderness ratios for vertically loaded walls. They represent a subjective assessment based on experience of extreme panel proportions, length ÷ thickness combined with height ÷ thickness, which did not exist in CP 111. Beyond these limits, there will be a rapidly increasing sensitivity to errors of design and construction, and sudden instability may occur. The form of the limiting dimensions for three- and four-sided support allows restrictions in one dimension to be traded against increases in the other, within an overall constraint of 50 times the effective thickness.

It is convenient to represent these limits graphically and Figure 30 gives the envelopes of allowable heights and lengths for walls of effective thickness 100 mm, 135 mm (equivalent to a cavity wall with two 100 mm leaves) and 190 mm. For typical storey heights of about 2.5 m the critical condition limiting length is provided in most cases by the general dimensional restriction of 50 t_{ef}, although for the thicker walls with three-sided support condition (a) (2) may control. Figure 30 also indicates the limits, derived from code Table 9, beyond which only two-sided support may be assumed ie panel shapes to which the method of clause 36.4.2 is restricted. For taller panels the limitations of (a) (1) and (2) and (b) (1) and (2) usually control the length, but for thin walls with four supported edges the height may be limited by 50 t_{ef}. For thicknesses greater than 100 mm, limitation of one dimension by 50 t_{ef} will automatically mean that the other dimension is also so limited.

36.4 Methods of design for laterally loaded wall panels

The developments leading to the design methods proposed here have taken place over little more than the last 10–15 years. The main lines of approach are distinguished by the part played by in-plane forces; it is convenient to follow that distinction here, dealing first with panel walls.

Much of the earliest attention to this problem was given in Scandinavia, particularly Sweden. Based on patterns of cracking observed at failure in panels with four sides supported, application of the yield-line theory developed for reinforced concrete slabs was proposed. However, due to the apparent inapplicability of yield-line assumptions to a brittle material like masonry, this approach did not at first find support here. The use of elastic analysis was explored as well but the development or application of both methods was hampered by the lack of experimental data. During the intervening years, this shortcoming has been remedied to a substantial extent by a considerable volume of laboratory testing at both full and reduced scale using an increasing range of bricks, blocks, mortars and boundary conditions.

Although knowledge of the load-deformation behaviour and ultimate strength of panel walls has thus increased, a more fundamental understanding of the internal mechanisms by which the moment of resistance of panels is developed has not been obtained yet. Re-examination of the available analytical approaches has swung in favour of the yield-line analogy, despite the continuing lack of rational justification. The approach finds favour because it enables advantage to be taken of the increased data on the orthogonal flexural strength properties of a range of types of masonry. It is possible to derive bending moment coefficients so that the ultimate strength is realised simultaneously in the orthogonal directions; appropriate account of boundary conditions may be taken as well. Nevertheless this approach can be justified only on the basis of correlation with experimental data and not on more fundamental grounds. For this reason, the basis on which code Table 9 has been derived is not stated explicitly. The only hint occurs in the suggestions in Appendix D for dealing with openings or irregularly shaped panels. As a major example of the justification for the design method recommended, Figure 31 shows the experimental results given by Haseltine, West and Tutt compared with the strength predicted by the design method.

It is worth contrasting the panel design method with the use of yield-line analysis for reinforced concrete slabs. In the latter case, design moments are calculated usually for two orthogonal directions based on the applied load and the edge restraints on the slab. Reinforcement is then provided to resist these design moments. In the present case, the moments of resistance in orthogonal directions are determined by the flexural properties of the masonry and the design moments induced by a uniform load are proportioned in the two directions according to the orthogonal ratio of strengths, as shown at the head of code Table 9. Therefore, equating moment and resistance in one direction automatically satisfies equilibrium in the orthogonal direction.

Figure 31 Lateral load experimental data and predicted strength.

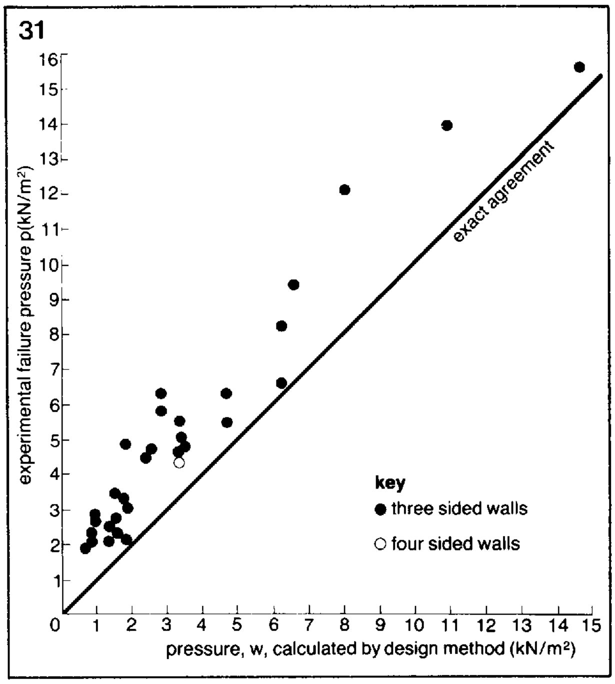
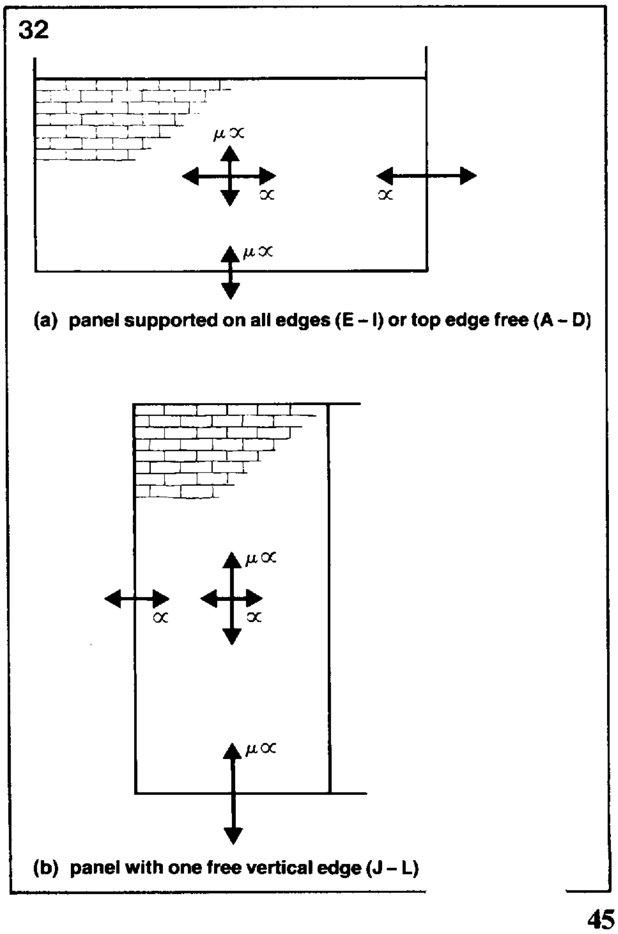


Figure 32 Distribution of panel bending moments.



The proportioning of bending moments in the two directions is illustrated in Figure 32, (a) corresponding to the diagram at the head of code Table 9 and (b) representing the correct form of C-shaped panel illustrated at J, K and L in the same table. The orthogonal arrows in the centre of the diagrams demonstrate that the ultimate moments are in the ratio of the parallel to perpendicular values of flexural strength. Continuity exists at an edge when the full moment in the appropriate direction can be resisted by the support.

Masonry usually exhibits greater resistance to bending in the horizontal direction, about a vertical axis, so the bending moment coefficients are expressed in relation to the horizontal direction. The expression for moment applies only to a uniformly distributed load so that when local increases occur eg at corners of buildings, it is necessary to equate the resulting maximum moment to that produced by an equivalent uniform load, and to design for that equivalent load. The load has been expressed as a characteristic wind load as this is the type of load which commonly will need to be considered, although occasionally design to resist bulk powders or granular materials may arise.

Due to the much lower flexural strength when failure is parallel to the bed joints, there are certain types of panel which are more sensitive to the strength in this direction. They are particularly panels with high ratios of height to length and with one vertical edge unsupported. In these cases, it will often be advantageous to take account of the reduction in flexural stresses afforded by compressive stresses due to the self-weight of the masonry or by other vertical loads of similar magnitude. For masonry of low flexural strength, a half-storey height may increase the effective characteristic strength in the parallel direction by as much as 50%. When such an allowance is appropriate, the flexural strength should be modified before dividing by the flexural strength for the perpendicular direction to obtain a new value for the orthogonal ratio μ^1 which should then be used in design to provide an appropriate bending moment coefficient α.

The original form of words in the code was intended to emphasise that the modification to flexural strength should be made on a compatible basis, ie either as design stresses or as ultimate stresses. If f_{ka} and f_{kb} are the flexural strengths in the parallel and perpendicular directions respectively, $\mu = f_{ka}/f_{kb}$ or $\mu = f_{ka}/\gamma_m \div f_{kb}/\gamma_m$ using design strengths. If g_d is the design vertical stress, the modified orthogonal ratio $\mu^1 = (f_{ka}/\gamma_m + g_d) \div f_{kb}/\gamma_m$ or $\mu^1 = (f_{ka} + \gamma_m \times g_d)/f_{kb}$ which corresponds to Amendment No. 2. It is worth noting that the numerator of μ^1 is identical to the effective flexural strength given in clause 35.5.3 for a free-standing wall. g_d should be determined using $\gamma_f = 0.9$.

Continuing this temporary notation for the orthogonal strengths, f_{kx} in clause 36.4.3 for a panel is equivalent to f_{kb}. However, when considering a vertically spanning wall with no restraint to its vertical edges only the weaker direction contributes to the design moment of resistance which is then based on f_{ka}. A contribution from vertical load is likely to be especially valuable in this case and f_{ka} may be modified in the manner just described. Although there may be restraints at top and bottom which will reduce the design moment below that applicable to a simply-supported panel, care should be taken to ensure that such restraints will continue to act throughout the design life of the panel.

The suggested allowances for flanges when calculating section moduli of piers enable the piers to be designed as vertical lateral supports. The intervening wall may be designed then as a panel wall with three- or four-sided support as appropriate. No distinction in flexural strength is made between solid and hollow blocks so the gross plan section should be used for the latter when calculating the section modulus.

Comment on clause 36.4.5 may be included here since it concerns only panel walls. During the early stages of experimentation, it seemed likely that separate recommendations would emerge for the use of wire ties and vertical twist ties. In the event, the difference appeared to be only one of degree, that is, it is reasonable to add together the moments of resistance of the two leaves of a cavity wall to determine the overall resistance, as long as the ties are strong enough to transmit the necessary forces. Clearly, wire ties will reach their ultimate capacity at lower loads than the same pattern of vertical twist ties, but nevertheless they will be adequate for a wide range of uses especially in domestic construction where their flexibility is necessary when brick and block leaves are used in conjunction.

There is some concern that butterfly ties, due to their method of manufacture, may have insufficient axial stiffness to transmit adequate forces without too much movement. While the concern appears to be reasonable, there is no evidence of problems in practice due to this aspect of performance and of course experimental walls have incorporated typical manufactured ties. The adequacy of the ties should be checked using the values of code Table 8 and the guidance in clause 36.2, including the value of 1·25 kN for double-triangle ties, and 0.5 kN for butterfly ties. The recommendation to add the resistance of the leaves, although reinforced by experiment, is difficult to justify analytically when the leaves comprise dissimilar materials or have differing edge constraints. Clearly, the forces in the ties will depend on the deflections of the walls, determined in turn by their effective stiffnesses; and in general the two leaves will not develop their ultimate moments simultaneously nor necessarily with identical fracture patterns.

It is suggested that the applied horizontal force is shared between the two leaves in proportion to their design moments of resistance. For a given section modulus the resistance is proportional to flexural strength which in general terms is proportional to stiffness, so that the proposition is reasonable. Again, it is difficult to justify theoretically because the proposal, when stated analytically, involves in fact a circular argument. However, for a design procedure, the last paragraph of 36.4.5 may be adopted to derive first the tie forces to be checked and then to check the strengths of the two leaves separately.

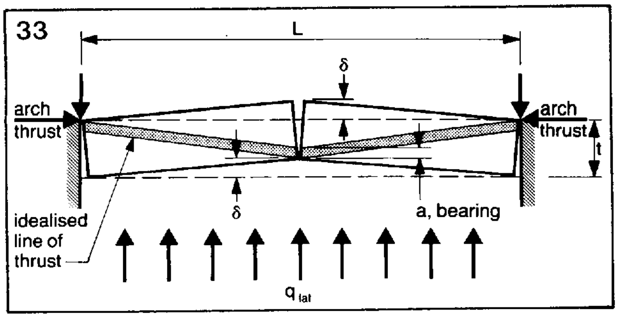


Figure 33 Horizontal arching mechanism.

36.4.4 Arching

Turning now to arching, the main experimental approach in justification of horizontal arching comprises tests on walls generally 2.7 m long and storey height built between massive abutments capable of resisting horizontal forces without significant displacement. The description of the design method can be amplified most usefully by reference to Figure 33 which shows the lines of arch thrust which are assumed to develop within the wall at failure and the effective bearing areas of these forces. Failure occurs in the arching mode when sufficient shortening of the two halves of the arch has occurred for the geometry to be overcome. It should be noted that when arching develops, that is when there is no possibility of in-plane movement of the ends of the wall, cracking may occur at a load substantially below that at which ultimate failure occurs. This condition of cracking corresponds approximately to failure of a panel wall having continuity over its vertical edges.

Under conditions of limiting equilibrium, the applied lateral load, derived by taking moments for half the wall about one end, is determined by the horizontal in-plane thrust developed and its bearing areas, which in turn determines the 'arch rise' or moment arm of the thrust. At failure, local crushing, and therefore shortening, occurs at the crown and the abutments. Therefore, in the face of only limited attempts to measure the thrusts directly, the thrust may be assumed to be determined by the compressive strength of the masonry f_k. Due to the local nature of the stresses, a 50% increase may be assumed following the recommendations of clause 34, bearing in mind that strictly speaking f_k should be determined for the loading direction parallel to the bed joints.

It is likely that in practice the bearing area will vary with the strength of brick which will determine the extent of redistribution of local stresses. Nevertheless calculation of the area necessary to sustain the assumed concentrated stresses suggests that a bearing equal to a tenth of the wall thickness is reasonable. The ultimate thrust is thus 1.5 f_k (t/10) per unit width (for a horizontally spanning wall, width in this expression means its height).

From consideration of moments the design uniform lateral load $q_{lat} = 8 \times$ design thrust $\times$ moment arm of thrust $\div$ (length)^2. If the length/thickness ratio is limited to 25 the deflection may be ignored, that is the correction to the arch is negligible. Substituting the appropriate expressions $q_{lat} = 8\left(\frac{3}{2}\cdot\frac{f_k}{\gamma_m}\cdot\frac{t}{10}\right)\left(\frac{9t}{10}\right) \div L^2$

which reduces, within 10% conservatively, to $q_{lat} = \frac{f_k}{\gamma_m}\left(\frac{t}{L}\right)^2$. In view of the assumptions about local bearing areas and f_k this approximation is sensible. q_{lat} is of course a design load so that to obtain a characteristic wind load, q_{lat} should be divided by an appropriate γ_f, namely 1.2 for this type of wall.

It must be emphasised again that the thrust can develop only if the supports can provide the necessary restraint. In practice there are likely to be relatively few circumstances in which full advantage can be taken of this form of action. In the absence of any tests it is not recommended to make any modification for self-weight or applied vertical load. Equally, there may be some panel walls in which in practice a degree of arching may contribute to the lateral strength but design should be based on either complete arching or only flexural strength (modified if appropriate).

36.5 Method of design for free-standing walls

No guidance is given in CP 111 for the design of these walls although their design as pure gravity structures is straightforward, even if not necessarily economical. In addition to this drawback there has been some concern that guidance on wind loads for such walls leads to values which are higher than experience suggests to be reasonable. Nevertheless, in this context satisfactory performance has meant simply that the overall factor of safety has not fallen below unity without necessarily implying correct design to a higher assumed margin of safety. Although this matter has not been resolved yet the ability to take account of a reliable flexural strength enhances the available design strength of a free-standing wall. As a hidden factor it may explain the apparent stability of walls designed on the basis of experience rather than calculation.

As such walls are usually exposed to the weather from both sides, the minimum mortar designation (iii) is given to accord with CP 121. A more cementitious mortar will often be desirable to increase both durability and flexural strength.

From the point of view of structural analysis, the design relationships are elementary. However, they have been included for completeness and because of the previous absence of guidance. The design moment is expressed in the form in which it will be used most commonly ie W_k for a wind load and Q_k for a parapet load, although other types of load are equally applicable. The single symbol γ_f is used because with wind, imposed and dead load acting combination 'c' in clause 22 applies. However, γ_f on dead load should not be 1.2 but 0.9 as discussed below. In the absence of imposed load, γ_f on wind load will still be 1.2.

The derivation of design moment of resistance recognises the importance of dead load as a force tending to restore equilibrium. In all cases in which it appears, whether in g_d design dead load per unit area or $n_w = (g_d \times t)$ design load per unit length, it is essential to use $\gamma_f = 0.9$ in the expression $g_d = \gamma_f \times G_k$. A restoring force, as opposed to a disturbing force, must be assumed to have a minimum value.

When flexural strength is invoked, g_d appears as a

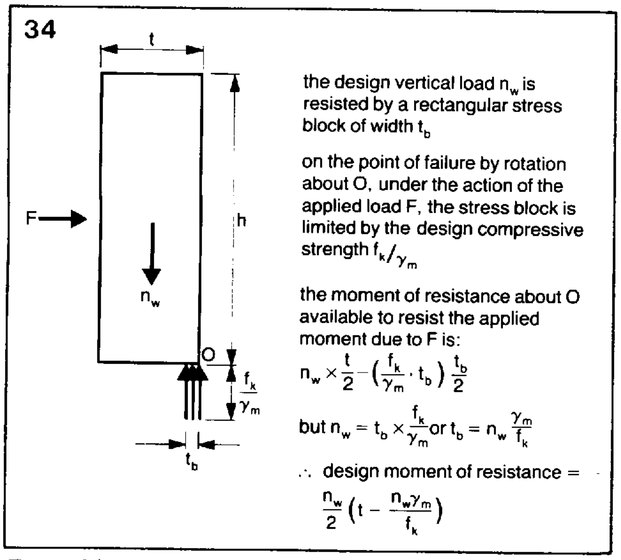


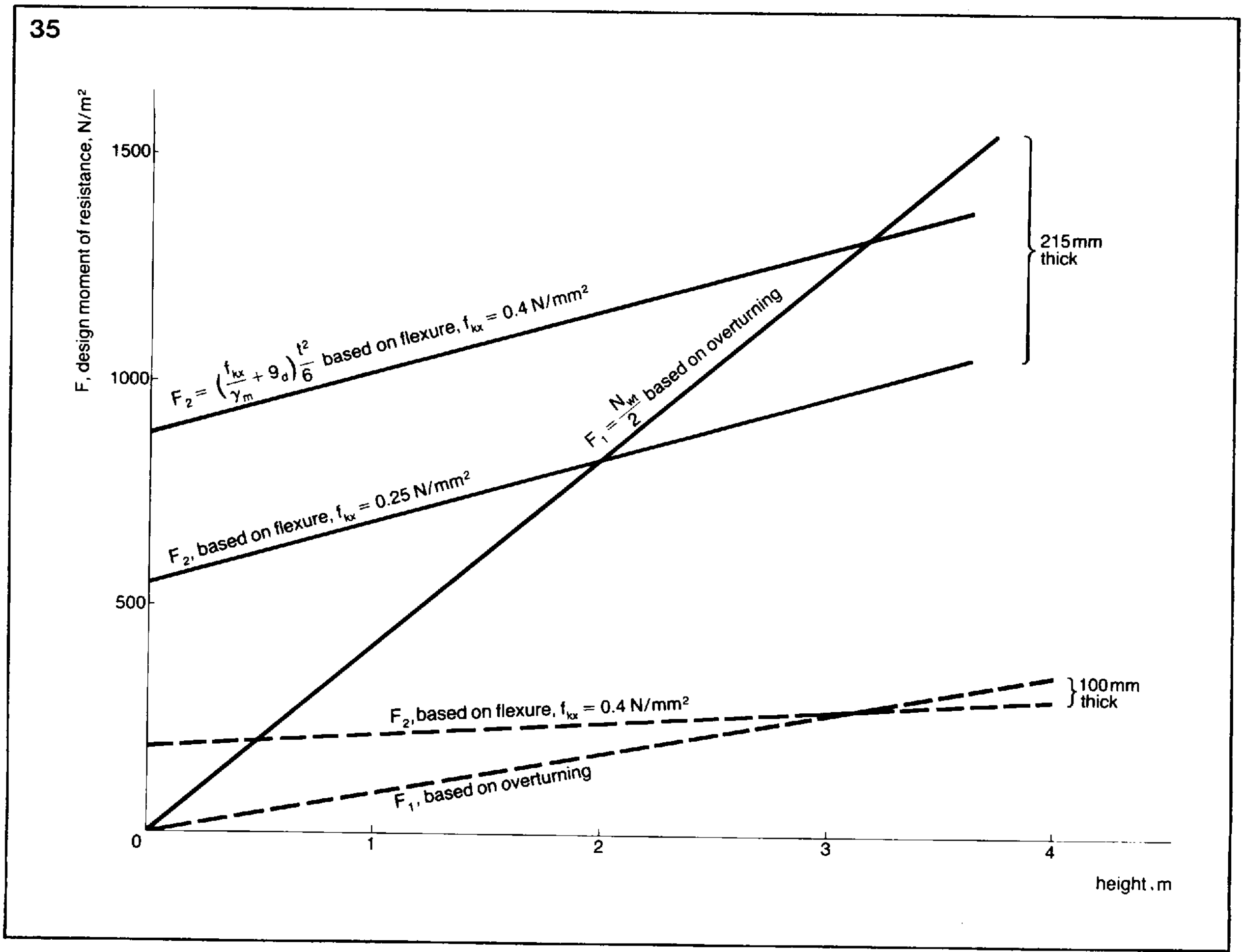
Figure 34 Design moment of resistance of free-standing wall without flexure.

modification to f_{kx}, as for panel walls in clauses 36.4.2 and 36.4.3. For practical walls this enhancement may be as much as 50%. If no flexural strength can be relied upon stability derives only from equilibrium between wind, imposed loads and dead load. Assuming that failure occurs by tilting, as of a rigid body, about an edge at a level at which the horizontal section is completely cracked, the design moment of resistance is simply $\frac{1}{2}n_w t$. A more sophisticated, and perhaps under some circumstances a slightly more realistic, assumption is that local crushing occurs in the vicinity of the edge over a depth determined by the compressive strength of the masonry. Assuming that the stress there is represented by a rectangular stress block, as in clause 32.2 and Appendix B for vertical eccentric load, reference to Figure 34 shows that the design moment of resistance should be reduced to $\frac{1}{2} n_w \left(t - n_w \frac{\gamma_m}{f_k}\right)$.

Considering the effect of n_w in parenthesis it might be argued that it should embrace a maximum g_d ie $1.6 \times G_k$, but different simultaneous values for γ_f can hardly be entertained. Equally pedantically, the narrow width of the ultimate stress block might suggest an overstress allowance on f_k. Whatever adjustment is made along these lines, the reduction in resistance is proportional to the ratio of design dead load to design compressive strength and will seldom make more than a few per cent difference except for particularly high or dense walls or very low strength masonry. It should be appreciated that this approach recognises that the wall will crack, and probably rock, prior to failure by overturning.

A comparison between the two approaches to moment of resistance is shown in Figure 35. It can be seen that increasing dead weight ie height of wall at constant thickness, provides a more rapidly increasing advantage to toppling resistance than to flexural resistance, although again the practical implications of this would be significant only for relatively tall walls. It does suggest that any reduction of flexural strength during the life of a wall will not necessarily lead to failure although a much reduced margin of safety may result.

Figure 35 Free-standing walls – design moment of resistance.



When there is a change in thickness of a wall cracking may occur at the level of the reduction. The stability of the relevant upper parts of the wall should be checked therefore, as well as that of the whole wall.

36.6 Retaining walls

Earth pressure is another type of loading for which statistical data is still not available. It may be assumed that, in the absence of other data, the values derived from Civil Engineering Code of Practice No 2 represent characteristic earth pressures. If they act as imposed loads, the partial safety factor should be taken as 1.6 (as in clause 22), but when earth pressure resists overturning a reduced value of 0.9 is appropriate. In the case of gravity retaining walls, γ_f for dead load should be taken as 0.9 for resistance to overturning, but 1.4 for checking ground bearing pressures. If flexure is to be relied upon, an appropriate value of γ_m will need to be selected from clause 27. The code for earth retaining structures uses both permissible stress and load factor approaches to safety factors. In the latter case, which is relevant to the failure of a gravity structure, an overall factor of safety between 1.5 and 2.0 is recommended, which is compatible with the above partial safety factors. Any pressure on the wall due to ground water should be considered as an imposed load but with a reduced value of γ_f of 1.2 to allow for variation in the depth of water. However, the resulting design pressure should not exceed that due to water equal to the full height of the wall. Resistance of a retaining wall to sliding is less amenable to limit state analysis at present and it is suggested that partial factors of unity are taken and that resisting forces should be at least double disturbing forces.

36.7 Foundation walls

This type of wall is distinguished from a simple retaining wall by the presence of vertical load, usually a superstructure. The lateral characteristic loading may be derived as described in clause 36.6 and the combined loading based on the partial safety factors in clause 22. The choice of design method between clauses 32 or 36 depends on the ratio of vertical to horizontal load and the uniformity of the lateral load.

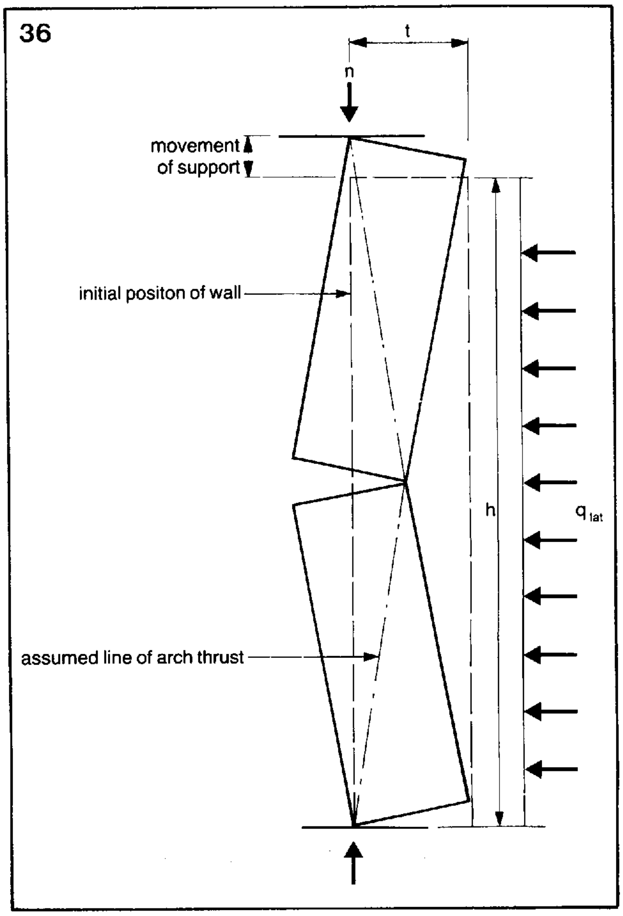
36.8 Design lateral strength of axially loaded walls and columns

Clause 32 has indicated the extent to which lateral loads may be treated as an effective eccentricity of vertical load when the latter is sufficiently high. In the wake of Ronan Point there was interest in defining conditions under which masonry could withstand much higher lateral loads and experimental work centred on walls capable of developing vertical arching, not unlike the horizontal arching described in clause 36.4.4. The main difference lies in the degree of resistance which can be provided realistically in the vertical plane of the wall, principally by the weight of the structure or possibly by vertical ties. Whereas some estimate had to be made of the arch thrust for horizontal arching, applied vertical loads may be calculated directly in vertical arching.

Tests have shown that failure occurs by formation of a three-pin arch with vertical movement of the in-plane loads. As the vertical load increases, local crushing starts to occur and disruption of the wall at failure becomes progressively worse. Nevertheless, even under these conditions, derivation of the ultimate lateral load from the arching mode of failure provides reasonable correlation with experimental failure loads. An idealised model of the three-pin arch at failure is shown in Figure 36. As with horizontal arching, the testing and analysis relate only to uniform lateral loads and there is not an explicit way of adapting the method to take account of non-uniform loads.

There are several end restraint and geometrical conditions which have to be satisfied for this method of analysis to be justified, and a separate partial safety factor has been incorporated explicitly in the design expression for q_{lat}. Rather than introduce yet another value, γ_m for compressive strength has been used even though, as indicated above, material properties have little influence on failure conditions. The presumption attending the choice of a reduced value of γ_m for appropriate quality control is that all the relevant conditions are controlled to a similar extent. The use of an unfactored characteristic load, for deriving n, is unusual but accommodated effectively by the presence of γ_m. Although γ_f, slightly less than unity, is introduced into the vertical load under accidential conditions, imposed load is included as well as dead load ie $0.95\ G_k + 0.35\ Q_k$. It would be more logical perhaps to base n on $0.9\ G_k$ for normal design purposes, but this method of design finds its principal application when considering design to resist accidental damage.

Figure 36 Idealised vertical arching.



The factors in code Table 10 are based on a limited amount of experimental work and to the extent that returns change the mode of failure from three-pin arching to yield-line type cracking the large values should be treated with reservation.

Jones L L and Wood R H,
Yield-line analysis of slabs.
Thomas & Hudson, Chatto & Windus London 1967.

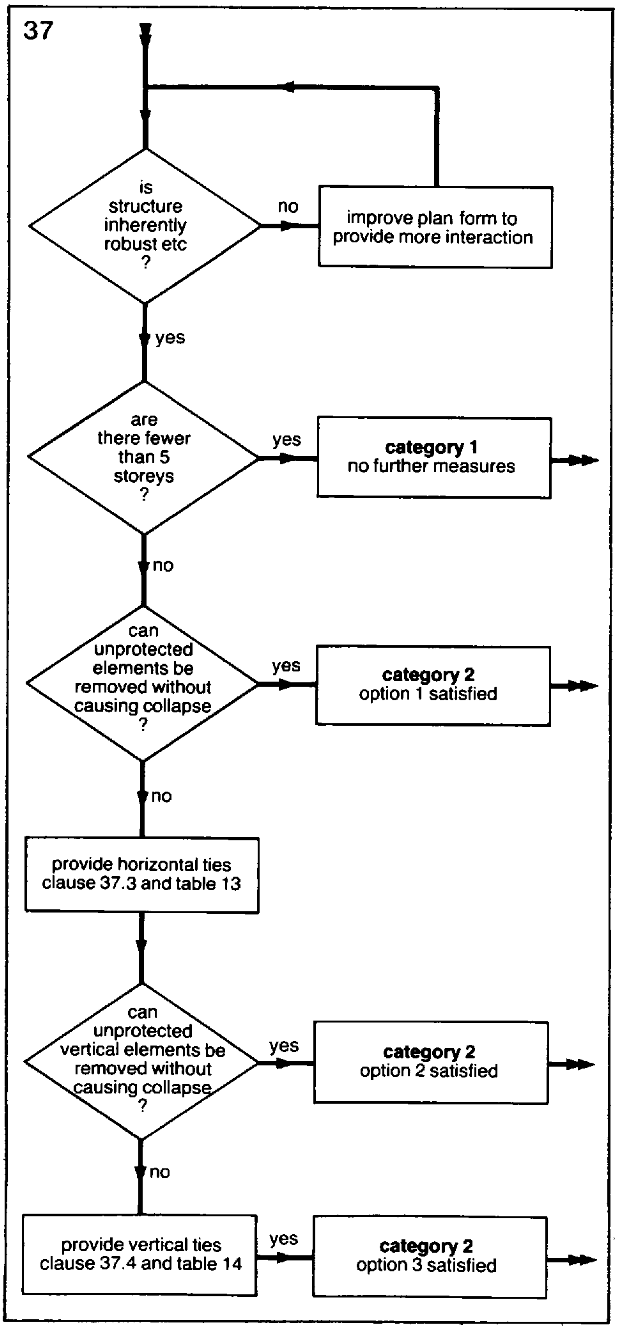
Anderson C and Bright N J,
Behaviour of non-loadbearing block walls under windloading.
Concrete v 10, n 9, September 1976.

Haseltine B A and Tutt J N,
Brickwork retaining walls.
BDA 1977.

Haseltine B A West H W H and Tutt J N,
The resistance of brickwork to lateral loading: Part 2.
The Structural Engineer v 55 n 10 1977.

Moore J F A, Haseltine B A and Hodgkinson H R,
Edge restraint provided by continuity of panel walls.
5th Int Brick Masonry Conf, Washington USA 1979.

Figure 37 Flow diagram for design to resist accidental damage.



SECTION 5 DESIGN : ACCIDENTAL DAMAGE

37. Design: accidental damage.

Because the recommendations in clause 37 appear for the first time in a code of practice they are presented in considerable detail. The specific guidance produced immediately after the collapse at Ronan Point related primarily to concrete structures and detailed recommendations were devised for CP 110 which was published in 1972. In 1969, general guidance for masonry structures was published by the Institution of Structural Engineers. Further detailed recommendations were made by the Brick Development Association and a substantial programme of testing gave rise to the recommendations of clause 36.8 as referred to in clause 37.1.1. Considerable experience evolved within the GLC and this combined information has formed the basis of clause 37. The general guidance in clause 37.1 is self-explanatory and is presented in tabular form in code Table 12. Use of this approach to the design of structures to resist accidental damage may be clarified nevertheless by means of the flow diagram in Figure 37.

The general precepts for design have been discussed in clause 20 which should be considered in conjunction with this section. The first question for all buildings is to establish that their layout and method of construction have been arranged to provide the best resistance to spread of damage. Although the code does not give any detailed guidance, the following features which specifically contribute to robustness, and are found in cellular construction, may be considered advantageous: avoidance of relatively thin or light-weight walls; limitation of floor spans; walls buttressed at both ends except for occasional free ends to minor internal walls; limitation of length of unbuttressed wall; and limitation of size of openings. There may be, of course, other functional or architectural requirements which conflict with these features and the designer must establish a desirable balance. As an example of the extreme differences possible, Figure 38 contrasts crosswall construction with a well buttressed cellular planform. The application of this philosophy, in common to all buildings, is emphasised by the format of code Table 12. Over and above these somewhat general exhortations for robustness, specific recommendations are made for buildings of five storeys and above, in line with Building Regulations.

The desirability of such a split has been the subject of much discussion. It is argued, on the one hand, that buildings below five storeys are sufficiently small for the spread of damage to be limited naturally to a reasonable extent by normal good design, and that the cost of introducing explicitly to Category 1 buildings the detailed recommendations for Category 2 would be unreasonable and of poor cost-effectiveness. On the other hand, it is argued that the division at 4–5 storeys is arbitrary, that the lack of specific guidance below 5-storeys places unreasonable responsibility on the designer and that a reasonable minimum provision of robustness is not secured.

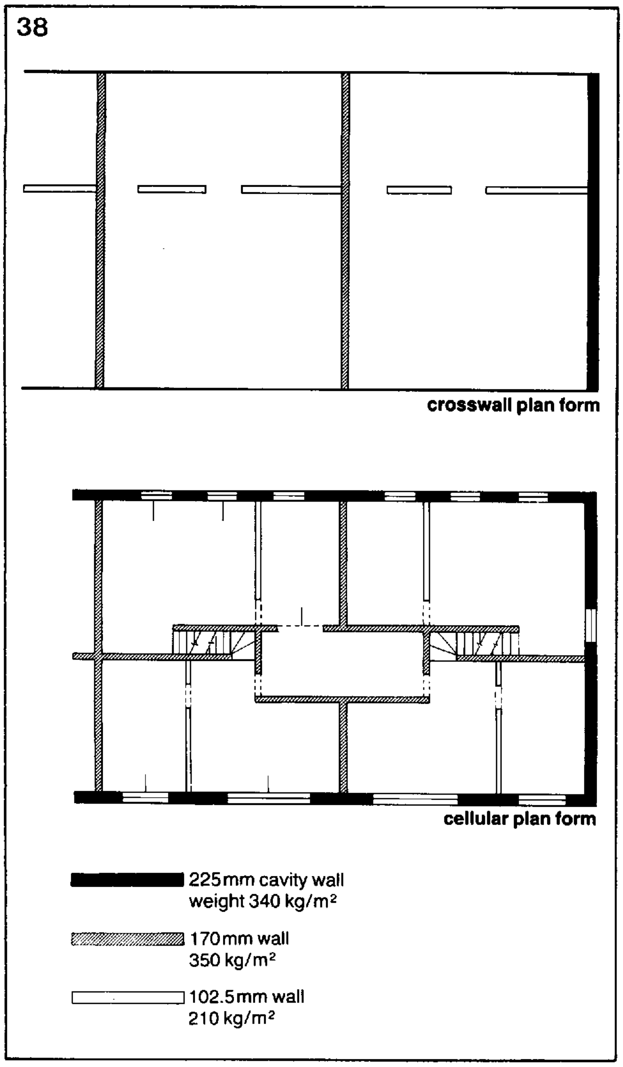


Figure 38 Comparison between crosswall and cellular plan forms.

A number of points are clear on both sides of the argument. The taller the building the more significant the structural aspects become as part of the total cost, and introducing the additional measures generally becomes relatively easier and cheaper. The possibility of extensive vertical progressive collapse is much greater in a taller building. In most cases, it is possible to design low-rise masonry buildings for normal loads in a manner which will provide adequate robustness. However, single-storey long span buildings appear to form a class of buildings which may be particularly sensitive to abnormal events unless particular care is given by the designer to their structural behaviour, but nevertheless no special recommendations have been made. The possibility of extensive horizontal progressive collapse, eg in a crosswall building, should not be ignored.

Once the general conception and layout of a building taller than four storeys have crystallised provisionally, it is necessary to examine the preliminary conclusions in the light of the three options suggested in code Table 12. The details of the clauses relating to these options are considered below but first interaction between the requirements of the options and the proposed layout may suggest alterations to the layout more amenable to a particular option. In many circumstances Option 3 will be selected because it prescribes horizontal and vertical tying without the need for any further consideration of structural behaviour. In other words, it is assumed that improved ability to accommodate local damage of any kind will result. To the extent that this approach is not related to any specific type of structure, it may be expected to be conservative, that is less economic to execute although possibly more economical of design effort.

Option 1 presents a more objective approach, sometimes known as the alternative path method, in which each loadbearing element is considered to be removed in turn, and the structure then checked for its ability to accommodate the loss. This more fundamental approach relies to a substantial extent on engineering judgement although guidance is given in clause 37.5 and code Table 11 for establishing realistic structural elements for removal. The designer must still decide what forms of structural action may be invoked to justify continued integrity and limitation of damage. In the light of previous arguments, it cannot be considered good practice to provide elements which are so vital to the structure that their removal cannot be tolerated without unacceptable risk to the rest of the structure. However, clause 37.1.1 gives guidance when these circumstances cannot be avoided or when removal of a wall or column need not be considered, the basis of loading being that in Building Regulation D 17. In fact, Option 1 is essentially a restatement of that Regulation without specific limitations on the meaning of 'collapse'. Many engineers will prefer the freedom of this more independent approach, especially when designing complex buildings.

Perhaps the most commonly adopted solution will be the recommendations of Option 2. This option combines the specific provisions of Option 3 with regard to horizontal elements with the more general approach of alternative paths of Option 1 for vertical elements. This option will find favour because buildings above 4 storeys will have concrete floors in which it is relatively easy to accommodate any additional horizontal ties, whereas vertical tying may present difficulties.

Although no conditions are specified to limit the application of the prescriptions for ties in Options 2 and 3 it is recommended that careful thought is given to the mode of action of ties. Only when it is clear that the spread of damage is likely to be contained by the provision of ties and that the ties themselves are not likely to have any adverse effects, should one of these options be adopted. In case of doubt a systematic application of the principles of Option 1 should be adopted. The three key features of these options, namely horizontal ties, vertical ties and load bearing elements, will be examined in more detail.

37.3 Horizontal ties

There are two main emergency actions expected of floors which can help to avoid the spread of damage. Firstly, when a vertical loadbearing element is lost, the floor should be able to bridge across the area of increased span, or in the case of loss of a corner wall, cantilever out, while still supporting the vertical structure above the floor. The second requirement is that the floor should be able to withstand the debris loading when collapse occurs from above. Both conditions envisage a floor developing two-way spanning action, or at least being able to span in either direction. In addition, the resulting deflection must be sufficiently limited to avoid too extensive disruption of walls above, particularly if the latter have limited ability to arch or span in their own plane. This clause is not specifically limited to concrete floors, but although timber floors may achieve the necessary bridging their deflection may be too large to limit damage above. The question of supporting debris is important for the top floor. The code recognises the impracticability of strengthening a lightweight roof to increase its spanning ability, and thereby implies that the debris resulting from disruption of a roof will fall to the floor below. Increased diaphragm action may be required as well to transmit lateral forces over greater distances to suitable shear elements.

These requirements could arise naturally from adoption of Option 1 and would have to be satisfied by the designer in an appropriate way. Code Table 13 provides specific recommendations at A and B for solving this problem. In doing so it follows closely the provisions of CP 110 but expresses them in a tabular and more graphic manner. However, these recommendations apply only above four storeys, not to all heights as in CP 110, because normal robust construction is considered to render them unnecessary for low-rise buildings. A particular implication for low-rise buildings is that pre-cast concrete floor units may not require additional tying provided that the units are adequately tied to walls and have large enough bearings on the walls (see clause 28.2.2 and Appendix C).

Various types of ties are considered and the force to be resisted is related to a basic force, F_t. A pressure of 34 kN/m² acting against a typical storey height wall can be equated to equal horizontal forces at the top and bottom of the wall of about 40 kN/m length of wall. This is the derivation of the minimum value of F_t, but F_t is also related to the number of storeys in an attempt to equalise the risk for all heights even though the risk of damage may be related to the size of building. Nevertheless, it is considered that no increase in F_t is necessary above 10 storeys, partly because a generally stronger structure is a natural concomitant of a greater number of storeys.

The peripheral tie force is related only to F_t, but the internal tie forces are also functions of dead and imposed load and floor span based on typical upper limits for domestic construction. The peripheral force is specified primarily to deal with loss of an external wall immediately below but it may serve to anchor the internal ties. Although the fixing requirements for the internal ties appear to include the possibility of their anchorage to the walls, it is doubtful whether this can be achieved conveniently in masonry, except in conjunction with vertical ties (clause 37.4 and code Table 14) which need to be anchored vertically to floor slabs. It would not be right to interpret horizontal ties as tying opposite masonry walls together, as might be the case in concrete panel construction, because unreinforced masonry walls would seldom be capable of resisting the implied moments imposed on them. However it is essential that walls or columns should not fail prematurely by loss of their horizontal connection at horizontal lateral supports. Therefore, at C and D in code Table 13, strengths of such connections are given in relation to F_t and the storey height. The normal practical method of achieving this connection will be in horizontal shear or by friction. Again, the function of this type of tying is not so much to attach the walls together as to ensure the lateral restraint conditions necessary for developing the vertical load capacity of the walls. When designing reinforcement for horizontal ties the calculated design tie force should be divided by the characteristic strength of steel to derive the cross-sectional area required. No partial factors of safety are involved in this calculation. Steel already provided to resist normal loads may be considered for these ties, the stresses due to the normal loads being ignored for this purpose only. The complete pattern of ties required is indicated in Figure 39.

37.4 Vertical ties

The horizontal tying to ensure a minimum level of integrity of floors and roofs is relatively straightforward. However, the actions which can be provided or enhanced by vertical tying are less obvious and perhaps less certain in their effectiveness. The following possibilities may be envisaged: (1) resistance to loss of horizontal lateral support to walls caused by uplift of floors subjected to explosive pressures; (2) maintenance of vertical loads sufficient to permit vertical arching of walls, as in clause 36.8; and (3) enhancement of interaction between wall and floor bridging an area of local damage.

In all cases, the provision of vertical ties to maintain the relative positions between superjacent floors could be advantageous. In the first case, safeguarding lateral support is complementary to horizontal tying at C and D in code Table 13 when a friction or shear connection is to be relied upon at the top or bottom of a wall. In the second case, lateral load resistance develops against the vertical thrust, provided normally by the dead weight of the structure. When explosive forces tend to nullify the dead weight the necessary precompression could be provided by strain in the ties as long as their extension is not sufficient for the wall to be pushed out. This type of action raises questions about the detailed response of the structure because the tying resistance must be developed at the same time as the lateral load develops on the wall, even though the dynamic behaviour of wall and floor may differ. In the third case the tying action could enable floors and walls to act as a composite deep beam of considerable capacity.

The design requirement for vertical ties given in code Table 14 is based on the case of vertical arching. Direct equation of an accidental pressure of 34 kN/m² to the expression for q_{lat} in clause 36.8 leads to the vertical force to be resisted by the ties, if γ_m is ignored ie, taken as unity instead of 1.05. The algebraic expression is a total vertical

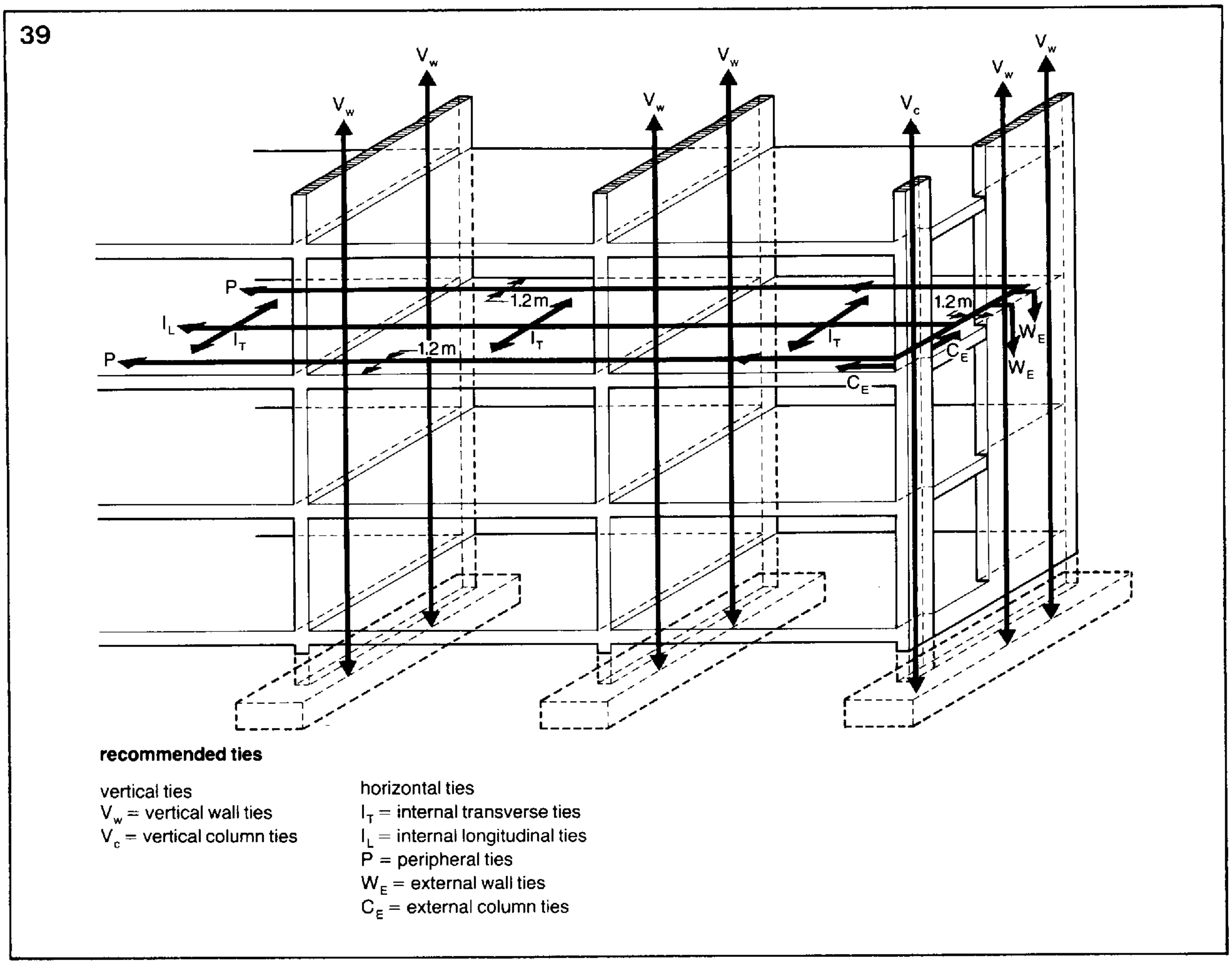


Figure 39 Full tying requirement.

force to be provided by ties at a convenient spacing, within a maximum of 5 m. The maximum force which would be required at this spacing is 1275 kN, compared with the minimum at 100 kN/m of 500 kN for which about 1200 mm² of steel cross-section would be necessary. The minimum force corresponds approximately to the minimum percentage steel requirement for vertical ties in plain concrete walls or the weight of a storey height wall.

The steel may be distributed at intervals along a wall, but it will be usually more convenient to concentrate it in pockets, where it may be also better protected against damage and corrosion. For a cavity wall, this means the leaf which carries the vertical load, usually the inner leaf. For a number of reasons, it appears desirable that ties, even though provided in every storey, should not de continuous but should be anchored independently for each storey. The failure of ties in one storey should not prejudice their action in adjacent storeys and certainly should not cause any spread of damage. A possible method of achieving this is by staggering the line of vertical ties in successive storeys.

37.5 Loadbearing elements

The designer is required in using Options 1 and 2 to consider the effect of removing individual loadbearing elements. This in turn requires assessment of the most likely extent of local structure which could fail under a range of accidental loads. In order to help the designer and

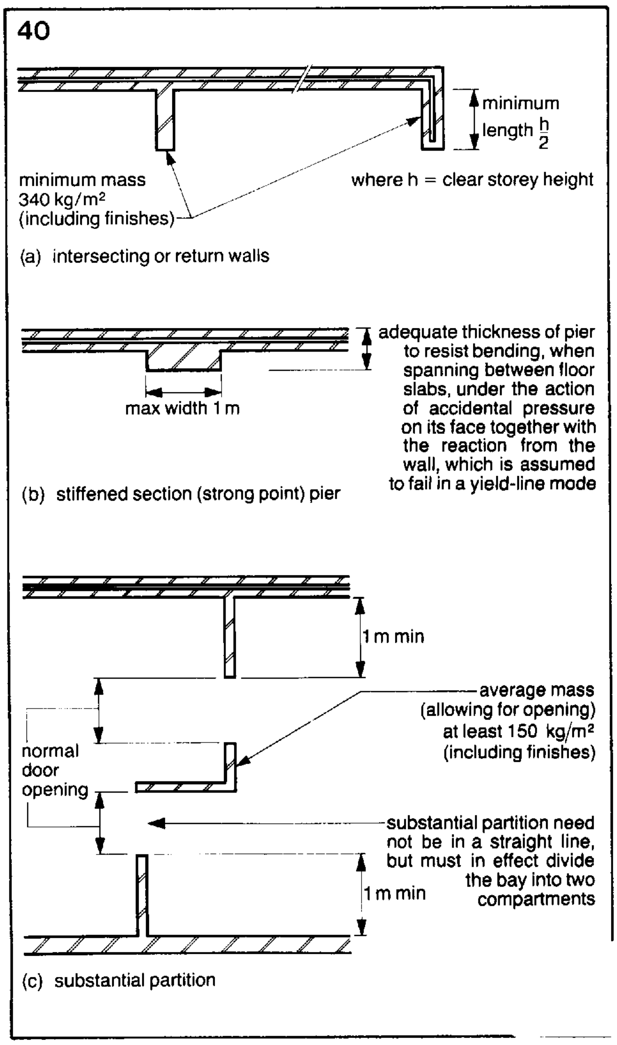


Figure 40 Lateral supports for loadbearing elements.

systematise the approach, code Table 11 defines the types of elements which it should be sufficient to consider. The extent of beams, columns, floors and roofs between supports is relatively clear, but the extent of vertical elements which might be lost is determined effectively by the adequacy of vertical lateral support or the presence of a free edge. Suitable supports are described in the clause at (a), (b) and (c) and are an extension to masonry of the similar requirements in CP 110.

Examples of the three types envisaged are shown in Figure 40 together with a sketch showing the derivation of the forces to be resisted by the lateral restraints. In type (a) the pressure on the supported wall on each side of the buttressing wall is assessed; in type (b) the pier or stiffened section has to resist that pressure as well as the direct pressure on the pier itself; and in type (c) the pressure is assumed to be confined to one side of the partition wall. 34 kN/m² is taken as the relevant pressure with a typical storey height of 2.5 m, the resultant force being approximated to a simple factored average value of F_t, F_t being in the range 40-60 kN for Category 2 buildings.

The Building Regulations *1976*.
HMSO 1976.

Korff J O A,
The overall appraisal of brickwork buildings.
BDA 1978. See also references to clause 20.

DESIGN

3

Chapter 2 has given the designer an insight into the background to the clauses of BS 5628: Part 1, their scope and explanations of the contexts in which they are used. Chapter 4 contains examples illustrating the use of the various parts of the code by considering the detailed design of two loadbearing masonry structures. This chapter acts as a bridge between the two, bringing together the essential design formulae and procedures explained in Chapter 2 and then used in Chapter 4. The aim is to provide a simple and concise reference section for the designer.

The code clauses have been arranged to follow the logical sequence of design procedures and the same general order is followed here.

DATA
(code clauses 6-18)

As with other structural materials the data to be used in the design of loadbearing masonry buildings must first be defined. This data includes the structural form of the building, a statement of the basis of design, the loads and forces to which the building will be subjected and the type and structural characteristics of the materials to be used.

Thus:
The structural units used should comply with the relevant British Standards, the principal of which are:

Calcium silicate bricks	BS 187
Clay bricks and blocks	BS 3921
Concrete bricks and fixing bricks	BS 1180
Precast concrete blocks	BS 2028, 1364.

Workmanship should be in accordance with CP 121: Part 1.

Characteristic loads should be obtained from the appropriate part of the loading code CP3, Chapter V, as follows:

Dead loads	Part 1
Imposed loads	Part 1
Wind loads	Part 2

Design should be in accordance with BS 5628: Part 1

DESIGN PROCEDURE
Stability
(code clause 20)

Having set down the design data, the next step is to consider the overall stability of the building. The following items need to be given consideration:

(*1*) *Layout and structural form*:
(a) The provision of sufficient return walls and their distribution.

(b) The interaction between the structural elements.

(*2*) *Resistance to overturning forces*:
(a) Resistance provided by wall complexes acting as vertical cantilevers.

(b) Resistance provided by shear walls (racking).

(c) Resistance provided by suitable bracing.

(*3*) *The overturning force, which is the greater of*:
(a) The design wind loading on the building.

(b) A uniformly distributed horizontal load equal to 1.5% of the total characteristic dead load above any level, see Figure. 6, page 19.

(*4*) *Stability design*:
(a) The masonry must be designed to resist the appropriate compressive loading resulting from the design overturning forces and the design vertical dead and imposed loads.

(b) The stress at the windward edge should be calculated. To avoid cracking no tension should be permitted and the load distribution may be assumed to be as in code Figure 3. However, in the ultimate limit state, a certain amount of tension may be permitted at the discretion of the designer as cracking is a serviceability limit state. See Figure 41.

(c) A check must be made to ensure that the characteristic shear strength of the masonry is not exceeded.

(d) If the building being designed is 5 storeys high or over (category 2) it must be designed to resist accidental damage. Buildings under 5 storeys (category 1) require no such additional consideration, beyond item 1 above.

Not only must the interaction between the structural elements, ie, the load path through the

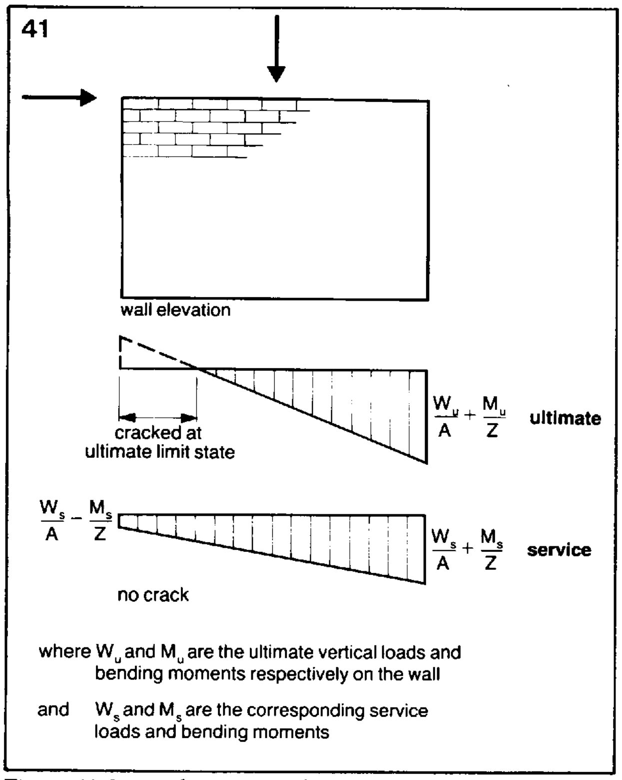


Figure 41 Stress diagrams under shear walls.

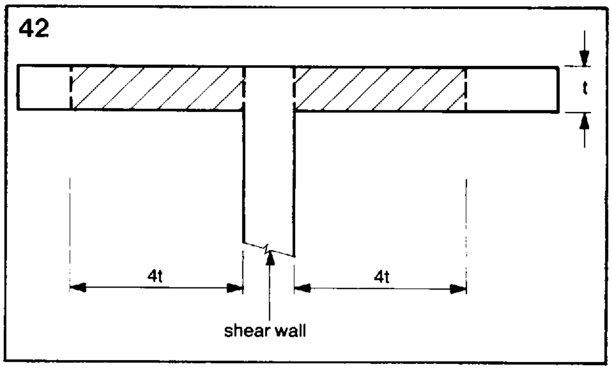


Figure 42 Length of flange which may be considered when flange is unrestrained.

structure to the foundations, (1) (b) above, be considered, but also the connections between the elements must be capable of transmitting the design forces. Appendix C is included in the code to assist the designer to satisfy this requirement.

Item 2(a) assumes that walls bend as vertical cantilevers and the code suggests in clause 30 that the horizontal force may be shared between the shear walls in proportion to their flexural stiffnesses. The code does not comment on the contribution to the stiffnesses that can be allowed for any fully bonded return walls perpendicular to the shear walls, but it seems reasonable to assume that the rules of code clause 36.4.3 relating to flange lengths may be adopted here; ie, the outstanding length of the flange from the face of the wall for the purposes of calculating the stiffnesses of the shear walls may be taken as:

(a) 4 × thickness of wall forming the flange where the flange is unrestrained, see Figure 42, or

(b) 6 × thickness of wall forming the flange where the flange is continuous, see Figure 43.

Item 4(a), design for compressive strength, constitutes the main body of the design process for masonry buildings.

The additional stability requirements for buildings of 5 storeys or over, mentioned in item 4(d), are discussed on page 50 under the heading Accidental Damage,

Having decided how overturning forces will be resisted by the chosen structural form, and having calculated the alternative overturning forces, it is necessary to consider items 4(a), (b), and (c). It is desirable at this stage to calculate the characteristic vertical loading for those walls on which the structure relies for stability, or at least on those walls which the designer considers will be critical. These characteristic loads can be presented best in tabular form keeping dead and imposed loads separate, to simplify the calculation of the design loads for the particular design case being considered. The use of the limit state approach in the code permits the degree of risk in a particular design case to be assessed by the use of different design load combinations.

Design load combinations
(code clause 22)

The design load is the sum of the characteristic loads (dead G_k; imposed Q_k; and wind W_k) multiplied by their appropriate partial safety factor for loads, γ_f. The code gives four combinations of design loads containing the appropriate numerical values of γ_f as follows:

(*a*) *Dead and imposed load*:
design dead load = $0.9G_k$ or $1.4G_k$
design imposed load = $1.6Q_k$

(*b*) *Dead and wind load*:
design dead load = $0.9G_k$ or $1.4G_k$
design wind load = $1.4W_k$ or $0.015G_k$ whichever is the larger.

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 = $1.2W_k$ or $0.015G_k$ whichever is the larger.

(*d*) *Accidental damage*:
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 = $0.35W_k$

Combination (d) for accidental damage is considered later. Each of the different combinations must be considered in the design process and that producing the most onerous loading condition used in the calculations. For a particular situation it may be obvious by inspection that one or more of the combinations will not be critical; for

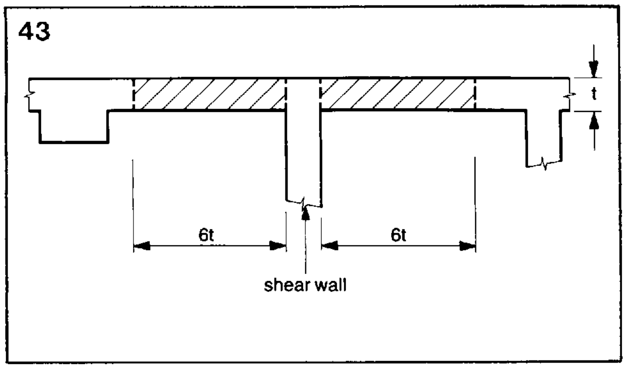


Figure 43 Length of flange which may be considered when flange is continuous.

instance when the element being designed does not carry wind loading, combinations (b) and (c) need not be considered.

In order to check whether tension occurs in a shear wall, combination (b) will be appropriate, the design dead load being taken as equal to $0.9G_k$. Similarly the same combination will give the maximum design horizontal load for consideration of shear. When calculating the maximum compressive load on a masonry element case (a), (b) or (c) may give the critical case. Combinations (b) and (c) are straightforward since in (c) no alterative values of γ_f are given and in (b) a design dead load of $1.4G_k$ will give the worst case in compression. However, when using combination (a) the situation is more complicated.

The code does not make clear whether the alternative γ_f values given for dead load should be used in conjunction with each other in the design of a particular structural element. Nor does it imply that γ_f on imposed loads should ever be taken as zero, although this makes engineering sense in obtaining the maximum eccentricity of load. It might, therefore, be assumed for a wall loaded from both sides that the code requires that the three cases shown in Figure 44 must always be considered: (i) giving the maximum vertical design load but minimum eccentricity, (ii) giving a high vertical design load combined with, when spans differ considerably, a large eccentricity and (iii) giving the minimum vertical design load but maximum eccentricity.

A mathematical exercise has shown that only cases (i) and (ii) in Figure 44 need generally be considered in design, since for (iii) to prove a worse case than (i) an unrealistic disparity in the length of the two spans onto the wall would be required; unfortunately a similar conclusion cannot be drawn for case (ii). The code does not require a case with zero imposed load to be considered. In the case of a wall or column loaded from above and one side it is only necessary to consider the maximum load condition shown in Figure 45.

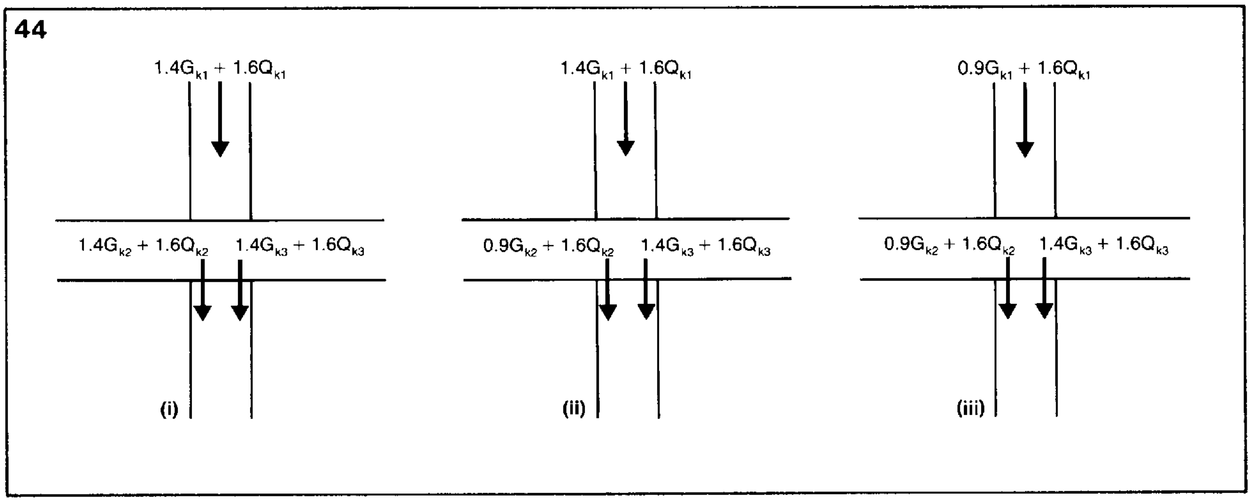


Figure 44 Alternative cases for load combination.

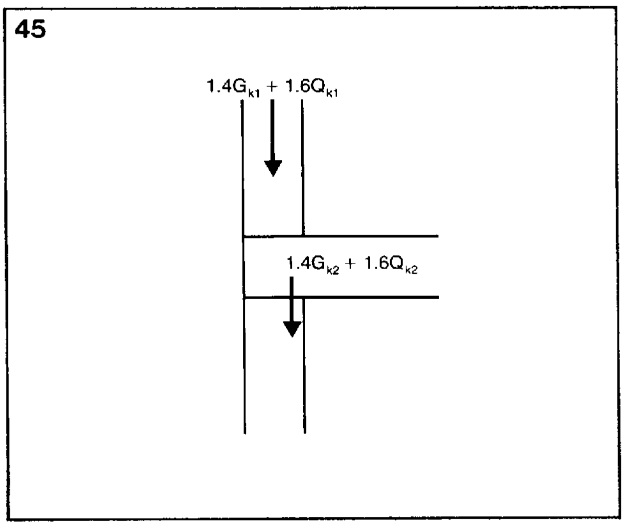


Figure 45 Loading condition to be considered when wall or column load from one side only.

Compressive strength
(code clauses 23, 28, 29, 30, 32, 34)

Masonry is most efficient as a structural material when used in compression, and so a major part of the design of loadbearing masonry is concerned with ensuring that the masonry has adequate compressive strength to carry its design vertical loads.

The design compressive strength of the masonry is related primarily to the characteristic compressive strength of the masonry unit/mortar designation combination, to the slenderness of the wall or column, and to the effective eccentricity of loading.

The relationship to be satisfied in design for compressive strength of masonry is:
design compressive strength of masonry $\geqslant$ design compressive loading.

It is normal practice to use this relationship to calculate the characteristic strength of masonry required for the structural element being considered. As this is the end result of the design procedure, the other factors governing the design compressive strength of masonry will be dealt with before consideration of the characteristic compressive strength.

Slenderness ratio
(code clause 28)

The slenderness ratio is defined in the code as the ratio of the effective height or length (h_{ef}) to the effective thickness (t_{ef})

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

Table 2 gives the effective heights and lengths of walls and effective heights of columns for the alternative lateral support conditions.

Table 2 Effective height/length of walls and columns

Element		Type of lateral support: Simple	Type of lateral support: Enhanced	Plane of lateral support
Walls	Effective height	1.0h	0.75h	Horizontal
	Effective length	1.0 l or 2.5 l_1	0.75 l or 2.0 l_1	Vertical
Columns (between openings in wall)	Effective height	1.0h	0.75 h+ 0.25h_1	Horizontal
Columns (general)	Effective height in plane with lateral support	h*	h*	

where:
h is vertical distance between lateral supports
h_1 is height of taller opening
l is length of wall, centre to centre of vertical supports
and l_1 is length of wall between vertical supports and vertical free edge.

**In a plane with no lateral support the effective height of a column is taken as 2h*

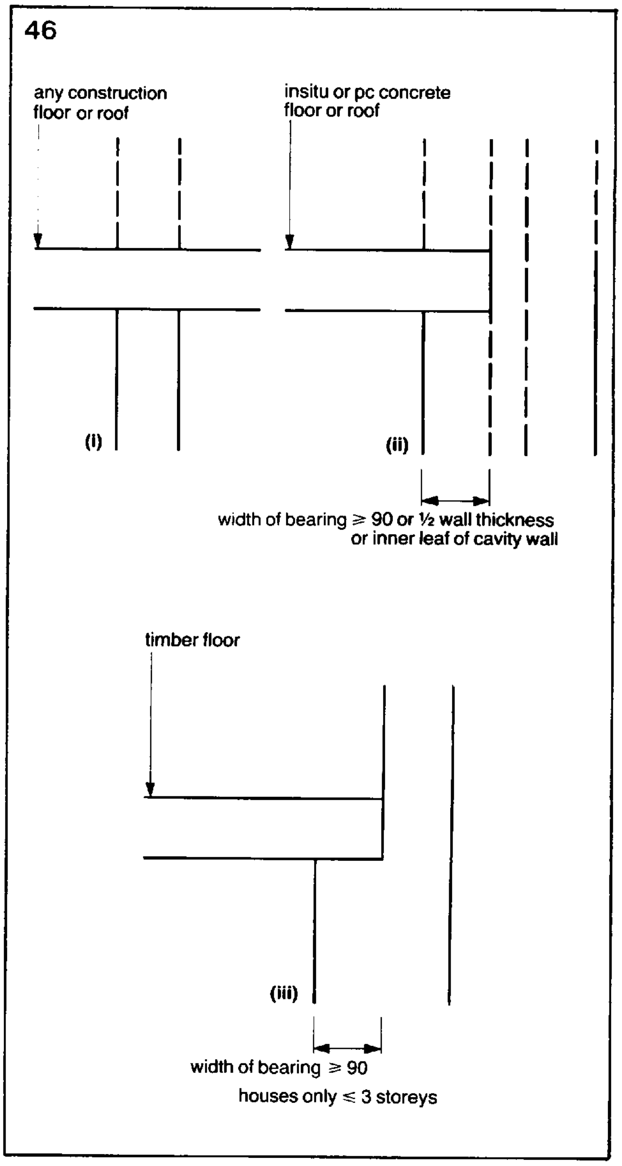


Figure 46 Enhanced resistance to lateral movement (horizontal lateral support).

Horizontal or vertical lateral supports must be capable of transmitting the sum of the simple static reaction from the applied design horizontal load and $2\frac{1}{2}\%$ of the total design compressive load that the wall or column carries at the level of lateral support to the elements of construction which provide overall stability, ie the shear walls or bracing. The $2\frac{1}{2}\%$ requirement need not be applied to the shear walls or bracing members themselves, since these are already designed to carry the appropriate proportion of 0.015 of the total characteristic dead load above the level of lateral support.

Appendix C to the code gives details of connections which may be considered to provide a horizontal lateral support with simple resistance to lateral movement. Enhanced resistance to lateral movement may be assumed in the cases shown in Figure 46.

Simple resistance to lateral movement for vertical lateral support may be assumed in the cases shown in Figure 47.

Whilst the code says that the only requirement for enhanced resistance is that the return or intersecting wall is properly bonded to the

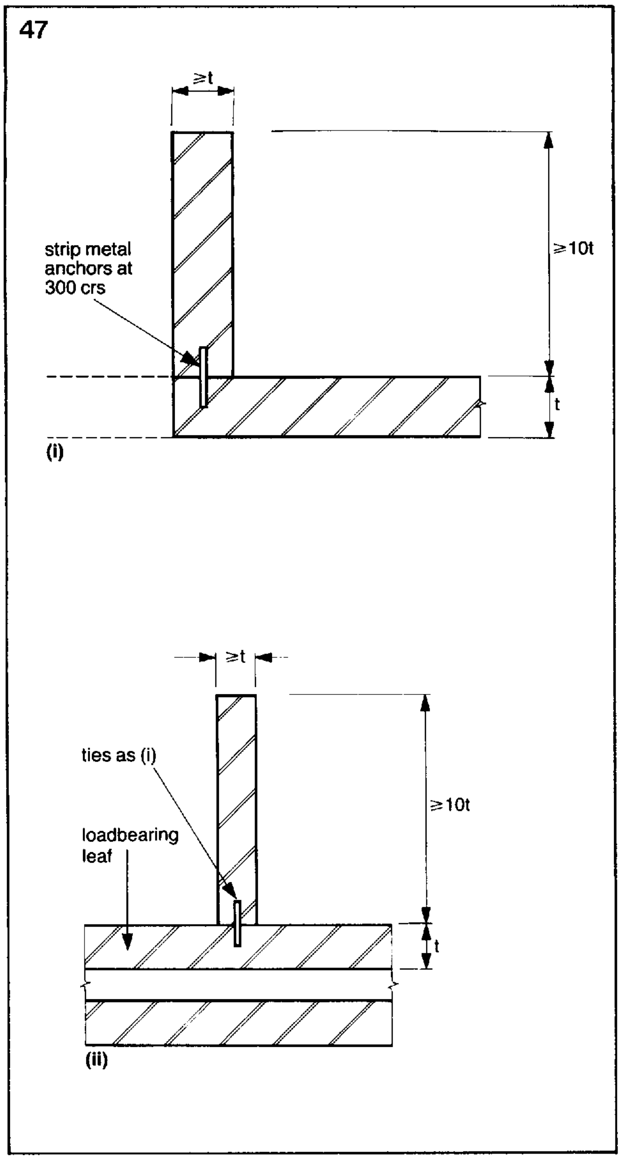


Figure 47 Simple resistance to lateral movement (vertical lateral support).

supported wall, it is reasonable to assume that the same restrictions on thickness and length of return wall as those for simple resistance should also be applied.

The effective thickness of a wall or column is obtained from code Figure 2 modified if necessary by the coefficient K for walls thickened by piers, obtained from code Table 5. Code Figure 2 and Table 5 are reproduced here as Figure 48 and Table 3 respectively.

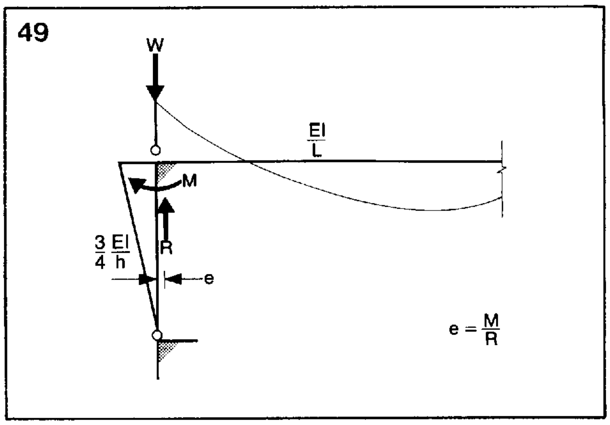


Figure 49 Calculation of eccentricity using moment distribution method.

Table 3 Stiffness coefficient, K, for walls stiffened by piers

Ratio of pier spacing (centre to centre) to pier width	Ratio $\frac{t_p}{t}$ 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 the table is permissible, but not extrapolation outside the limits given.

When a wall is stiffened by intersecting walls, such that the ratio of the distance centre to centre of the intersecting walls to the thickness of the stiffened wall is less than 20, the stiffness of this wall may be modified by the coefficient, K; the pier thickness t_p may be assumed to be 3 times the actual thickness of the stiffened wall or leaf.

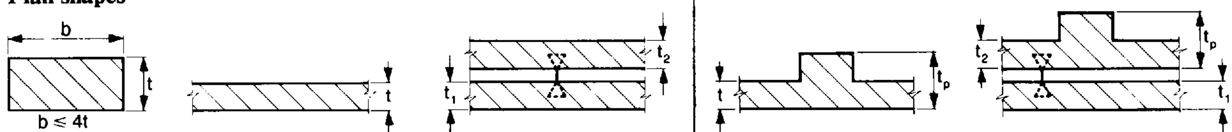
Eccentricity at right angles to wall
(code clause 31)

The code suggests that preferably the eccentricity should be calculated. The most obvious way of doing this is to use the moment distribution method. Using the conservative assumption that a pin joint exists at the bottom of a masonry wall or column, the eccentricity of loading being considered zero at this point (see code Appendix B), a moment may be calculated at the top of the lower masonry element and this can be equated to a load and eccentricity, as shown in Figure 49.

Figure 48 Effective thickness of columns and walls.

48

Column	Single leaf wall	Cavity wall	Walls stiffened by piers Single leaf	Cavity
Plan shapes				



Column	Single leaf wall	Cavity wall	Walls stiffened by piers Single leaf	Cavity
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 x 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 3

Similarly any other analytical method may be used to calculate the design moment in the masonry wall or column and hence the effective eccentricity.

The code also permits, at the discretion of the designer, the assumption that the loads act at one-third of the depth of the bearing area from the loaded face of the wall, as shown in Figure 50(a).

Figure 50 Notional eccentricity; solid wall loaded from one side.

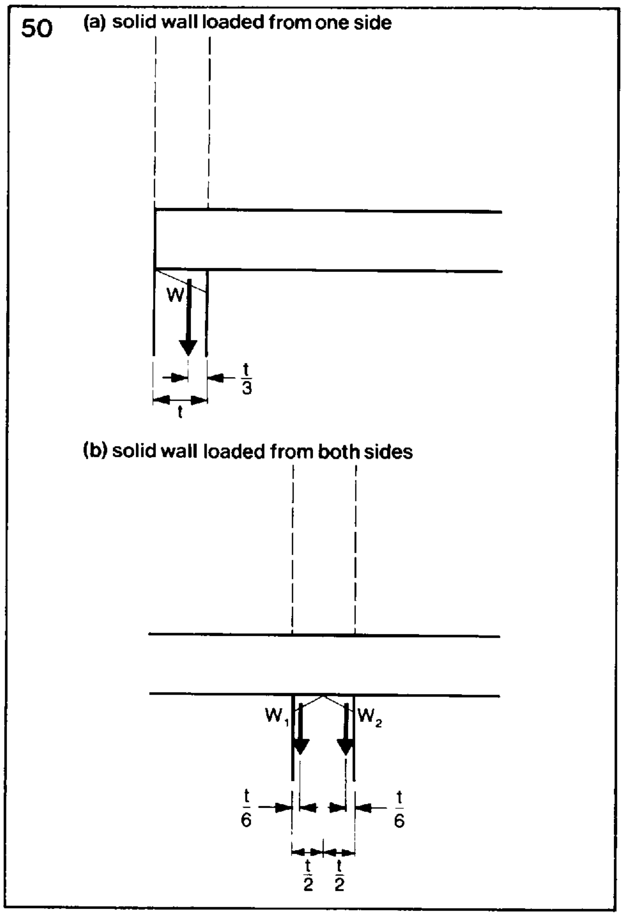
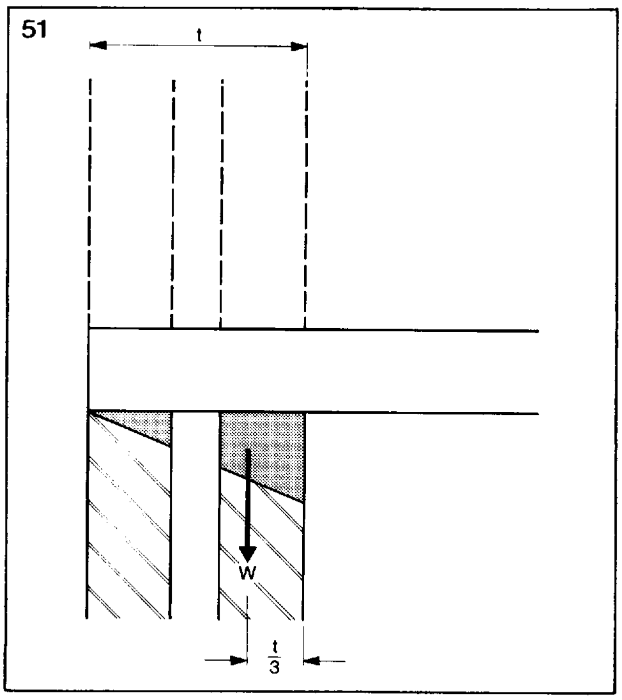


Figure 51 Assumed effective eccentricity of loading; cavity wall with both leaves loaded.



Where a uniform floor is continuous over a wall, a similar assumption for the eccentricities may be made as shown in Figure 50(b). As mentioned above, the conservative assumption should be made that the eccentricity of loads from above is zero.

When calculating the eccentricity of loading on a cavity wall which has both leaves loaded, the relative stiffness of the wall may be taken as the sum of the stiffnesses of the individual leaves. When only one leaf is loaded, it is reasonable to assume, provided wall ties are incorporated in accordance with the code, that the wall stiffness may be taken as two thirds the sum of the individual leaf stiffnesses, or the stiffness of the loaded leaf whichever is greater.

Corresponding to the simplified assumption of the load acting at one third the depth of the bearing area for a solid wall, the assumption shown in Figure 51 may be used for external cavity walls where both leaves are loadbearing.

In a cavity wall if the calculated or assumed net eccentricity of the applied vertical load, represented by the eccentricity of the reactive force R in Figure 52(a) and (b), is between the centroids of the two leaves as shown, the leaves may be designed as individual walls carrying the statically equivalent loads W_5 and W_6 as shown in Figure 52(c).

The benefit of the increase in stiffness of each leaf by virtue of their mutual connection is allowed for by using the slenderness ratio of the full cavity wall for the design of each leaf.

Design vertical load resistance
(code clause 32.2)

The tendency of a masonry wall or column to buckle increases with increasing slenderness ratio and eccentricity, and thus the load carrying capacity is reduced. This is allowed for in design by the application of a capacity reduction factor β obtained from code Table 7. The value of β is related in code Table 7 to the slenderness ratio and the eccentricity at the top of the wall and takes into account an additional eccentricity due to the buckling deflection. The derivation of β is given in code Appendix B.

The design vertical resistance of a wall is given by:

$$\frac{\beta\, t\, f_k}{\gamma_m} \text{ per unit length}$$

and of a column by:

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

where: t is the thickness of the wall or column
b is the width of the column
γ_m is the partial safety factor for material strength
f_k is the characteristic compressive strength of the masonry

γ_m is obtained from code Table 4, (Table 4 below). Whilst normal categories of manufacturing

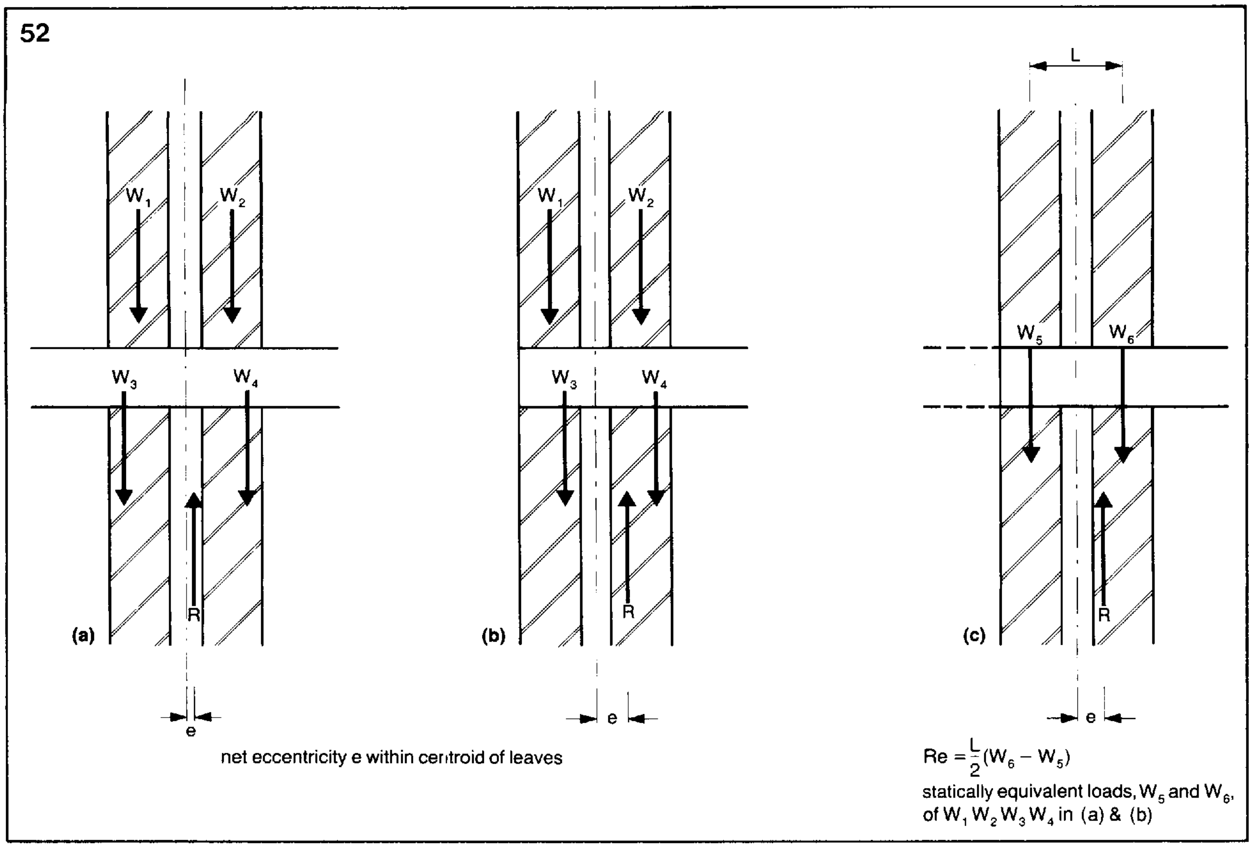


Figure 52 Eccentrically loaded cavity walls.

and construction control give a value of 3.5 for γ_m from code Table 4 it should be noted that an increase in loadbearing capacity of approaching 30% can be made by ensuring that the requirements of the special categories of manufacturing and construction control are achieved, leading to a γ_m value of 2.5.

Table 4 Partial safety factors for material strength, γ_m

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

The characteristic compressive strength of masonry, f_k, is usually the end product of this part of the design process, its calculation confirming that either the masonry units and mortar designation being proposed are adequate or enabling suitable units and mortar to be chosen.

In certain circumstances however the value of f_k is modified by factors as follows:

(a) Walls and columns of small plan area – If the horizontal *loaded* cross-sectional area, A, of the wall or column is less than $0.2m^2$ then the value of f_k must be multiplied by $(0.70+1.5A)$
(b) Narrow brick walls – If a brick wall, or the loaded inner leaf of a brick cavity wall, is narrow ie, of thickness equal to the width of a standard format brick, the value of f_k may be multiplied by 1.15
(c) Modular brick walls – If a brick wall is built using modular bricks the value of f_k may be multiplied by:
(i) 1.25 if the wall is narrow or
(ii) 1.10 for other thicknesses

Thus, the design vertical load resistance becomes:

(a) walls of small loaded plan area

$$= \frac{\beta t\,(0.70+1.5A)\,f_k}{\gamma_m}$$

columns of less than $0.2\ m^2$ loaded plan area

$$= \frac{\beta b\,t\,(0.70+1.5A)\,f_k}{\gamma_m}$$

(b) narrow walls

$$= \frac{\beta t\ 1.15 f_k}{\gamma_m} \text{ per unit length}$$

(c) walls in modular bricks:
(i) narrow walls

$$= \frac{\beta t\ 1.25 f_k}{\gamma_m} \text{ per unit length}$$

(ii) other thicknesses

$$= \frac{\beta t\ 1.10 f_k}{\gamma_m} \text{ per unit length}$$

When in (a) above, the wall or column is of cavity construction and only one leaf is vertically loaded, plan area 'A' should be taken as the area of the loaded leaf. It should be noted that the modification to f_k in the formulae given in (a) may occur in combination with (b) or (c), should they also apply.

Equating the above formulae to the design vertical loads for the wall and column elements being considered enables the minimum values of f_k to be calculated. From the value of f_k obtained, the appropriate brick or block strength and mortar designation can be selected from code Table 2 which is in four parts, reproduced here as Tables 5, 6, 7 and 8. Table 5 is used when masonry is constructed with standard format bricks. Table 6 when the masonry is constructed with structural units with a ratio of height to least horizontal dimension of 0.6 (brick size). Table 7 when the masonry is constructed with structural units, other than solid blocks, with a ratio of height to least horizontal dimension of between 2.0 and 4.0 (most normal hollow clay or concrete blocks) and, finally, Table 8 when the masonry is constructed with solid concrete blocks with a ratio of height to least horizontal dimension of 2.0 and 4.0 (most normal solid blocks).

Table 5 Characteristic compressive strength of masonry f_k, in N/mm², 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

Table 6 Characteristic compressive strength of masonry, f_k, in N/mm², 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

Table 7 Characteristic compressive strength of masonry, f_k, in N/mm², 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

Table 8 Characteristic compressive strength of masonry, f_k, in N/mm², 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

For units with height to least horizontal dimension ratios of between 0.6 and 2.0, f_k is obtained by interpolation between Table 6, and Table 7 or 8, whichever is appropriate. It should be noted that solid in the context of Table 8, means without cavities and not 'solid' to BS 2028/1364.

To simplify selection of brick class and mortar designation when using code Table 2(a), the Table is also shown in graphical form as code Figure 1. Amendment No. 2 now gives code Tables 2(b), (c) and (d) in graphical form as well.

It is usual to rationalise the class or strength of masonry unit and mortar combination required in various parts of a particular building for reasons of economy, and to simplify construction, thus reducing the risk of the wrong unit or mortar being used in a particular situation.

There are four further design considerations which may affect the required value of f_k and therefore the choice of unit strength and mortar designation. They are as follows:
concentrated loads
shear strength
walls subjected to lateral load and
accidental damage,
all of which are considered in the following sections.

Concentrated loads
(code clause 34)

To ensure that the chosen characteristic compressive strength of the masonry, f_k, is adequate to cope with the local effects of concentrated loads due to beams, lintels or padstones etc, the effect of the concentrated load, combined with stresses due to other loads, should be checked at the bearing and at a distance 0.4 h below the bearing, where h is the clear height of the wall. The code permits an increased local design bearing strength at the bearing, depending upon the type of bearing. It also permits the load to be dispersed at 45° through the masonry for the purpose of checking the design strength at 0.4 h below the bearing, at which level no increase is allowed.

The bearing type applicable to the particular case can be chosen from code Figure 4. Code Figure 4(a) represents the limiting dimensions of bearing area for which the local design bearing strength may be increased by 1.25. Bearings of this type may be designed for a local design strength of

$$\frac{1.25\,f_k}{\gamma_m}$$

If the bearing type corresponds to the code type 4(b), having a more limited bearing area, use may be made of a higher local design strength of

$$\frac{1.5\,f_k}{\gamma_m}$$

When a spreader beam is incorporated under a concentrated load as in code bearing type 4(c), the local design bearing strength may be taken as

$$\frac{2.0\,f_k}{\gamma_m}$$

The stress distribution under the spreader must be based upon an acceptable elastic theory. The code does not give any guidance regarding this, other than to indicate a possible shape of stress diagram in code Figure 5(b). Possible maximum design stresses may be arrived at using:

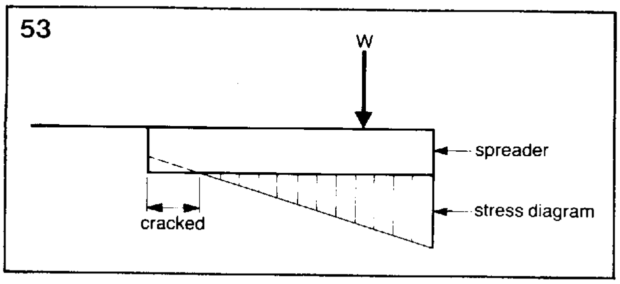


Figure 53 Spreader beam: $\frac{W}{A} \pm \frac{M}{Z}$ *approach.*

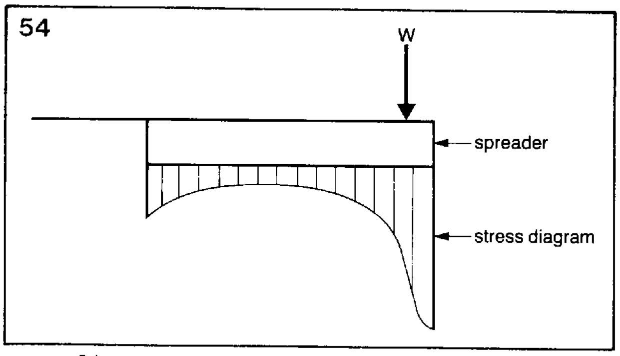


Figure 54 Spreader beam: Timoshenko's approach.

(a) the straightforward $\frac{W}{A} \pm \frac{M}{Z}$ approach. This gives a stress diagram that implies that cracking would occur under the inner part of the spreader if its length exceeds three times the distance from the line of action of the load to the end of the wall, as shown in Figure 53. The diagram would, of course, be modified by any other uniformly distributed loads acting on the wall form above.

(b) Timoshenko's analysis for the bending of bars on elastic foundations. Strictly this method is applicable only if the spreader is of infinite length, but it does result in a stress diagram (Figure 54) that is similar to that shown in code Figure 5.

When checking the effect of the concentrated load at a distance of 0.4h below the bearing, see code Figure 5, the design compressive strength should be calculated in accordance with code clause 32, (pages 60 and 97 in this book).

Shear strength
(code clauses 25, 27.4 and 33)

Walls subjected to horizontal forces acting in their plane must be checked for shear strength. Shear in laterally loaded walls is considered in the section dealing with such walls.

From the design horizontal forces obtained in the stability analysis the shear stress, v_h in walls resisting overturning may be calculated. The shear load is assumed to act uniformly over the horizontal cross-sectional area of the 'web' of the wall. The area of any flanges is not included since it can be shown analytically that the shear stress distribution across a section concentrates the shear load on the web, as in steelwork and reinforced concrete design.

When considering shear the following relationship must be satisfied:

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

where

f_v is the characteristic shear strength which equals $0.35+0.6g_A$ N/mm², up to a maximum of 1.75 N/mm², for mortar designations (i) (ii) or (iii) or $0.15+0.6g_A$N/mm² up to a maximum of 1.4 N/mm², for mortar designation (iv).

g_A is defined as the design vertical load per unit area of wall cross section, due to the vertical dead and imposed loads calculated from the appropriate loading case. However, the appropriate loading case, giving the maximum design horizontal load, and hence shear load, together with minimum vertical load will always be load case (b), see page 57. No imposed load is included. Thus $g_A=0.9G_k$

If g_A varies along the length of a wall, its average value may be used to determine the characteristic shear strength , being calculated either directly or by integrating the load distribution diagram and dividing by the wall length.

γ_{mv} is the partial safety factor for masonry strength for shear and is given in the code as 2.5, provided the mortar is not weaker than designation (iv). When considering the effects of misuse or accident a value of 1.25 may be used for γ_{mv}.

Walls subjected to lateral load
(code clauses 22, 24 and 36)

Clause 36 of the code deals with the design of wall panels which are subjected to lateral loading. Such design is generally only necessary when either the lateral load is the predominant load on the wall, and there is little vertical load other than self-weight, or the wall has to resist accidental damage. The latter case is dealt with on page 68.

The designer must first decide what restraint is offered to the panel by its supports and satisfy himself that the supports are capable of resisting the forces transmitted to them by the panel. Edge restraint will depend upon continuity over the support, the restraining influence of any vertical loading on the wall, the presence and type of dpc at the supports and the adequacy of the connection to the supports. If the panel is to be designed as an arch, the strength of the abutments must be adequate to develop the in-plane forces. Code Figures 6 and 7 give some guidance on the assessment of edge restraint. The adequacy of the ties can be checked using code Table 8, reproduced below as Table 9, and the characteristic compressive strength, given in the code, for double triangle and wire butterfly ties in mortar designation (i), (ii) and (iii) of 1.25 kN and 0.5 kN respectively. The partial safety factor, γ_m, to be applied to the strength of wall ties is 3.0, although this may be halved when considering misuse or accidental damage. In cavity walls especially it is necessary

Table 9 Characteristic strengths of wall ties used as panel supports

Type	Characteristic strengths of ties engaged in dovetail slots set in structural concrete Tension kN			Shear kN
Dovetail slot types of ties				
(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***
	Mortar designations			**Mortar designation**
	(i) and (ii) kN	**(iii)** kN	**(iv)** kN	**(i), (ii) or (iii)** kN
Cavity wall ties†				
(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

**Applicable only to cases where shear exists between closely abutting surfaces*
†*See BS 1243: 1978*

to ensure that lateral forces can be transmitted to the supports if wall ties are being relied on for this purpose.

Limiting dimensions
(code clause 36.3)

The code gives limiting dimensions for panels or walls subjected to lateral loading for mortar designations (i) to (iv) as follows:

(*a*) *Panel supported on three edges:*

(1) two or more sides continuous:
height × length equal to $1500t_{ef}^2$ or less

(2) all other cases:
height × length equal to $1350t_{ef}^2$ or less

(*b*) *Panel supported on four edges:*

(1) three or more sides continuous:
height × length equal to $2250t_{ef}^2$ or less

(2) all other cases:
height × length equal to $2025t_{ef}^2$ 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}.

Design of laterally loaded wall panels
(code clause 36.4)

The code gives two approximate methods for designing a laterally loaded panel:
(a) as a panel in bending supported on a number of sides.
(b) as an arch spanning between suitable supports.

Method (a)

The method does not allow for in-plane forces but does permit some allowance to be made for precompression due to self-weight or applied vertical loads in suitable circumstances. It also takes account of the difference in strength of masonry in orthogonal directions and different edge support conditions.

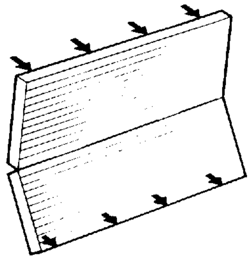
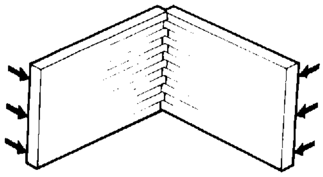
To some extent the design process is by trial and error, since it is often necessary to assume a masonry unit/mortar designation combination and then to check the structural adequacy of that combination.

For bending about a vertical axis the design moment per unit height
$= \alpha W_k \gamma_f L^2$

where: α is the bending moment coefficient
γ_f is the partial safety factor for loads
L is the horizontal dimension of the panel
W_k is the characteristic wind load per unit area.

α is obtained from code Table 9 for the appropriate panel shape and edge conditions. To obtain α, as well as knowledge of the geometry of the panel, the orthogonal ratio of the masonry, μ, must be known. This ratio is defined as the ratio of the flexural strength of masonry when failure is parallel to bedjoints to that when failure is perpendicular to the bed joints. The characteristic flexural strength of masonry, f_{kx}, for the two orthogonal directions is obtained from code Table 3, reproduced below as Table 10. For clay brickwork μ is sensibly constant at 0.35 so that prior assumptions of brick and mortar need not be made; however, for concrete blockwork μ varies with block strength and an initial guess is necessary to obtain α.

Table 10 Characteristic flexural strength of masonry, f_{kx}, N/mm²

	Plane of failure perpendicular to bed joints			**Plane of failure parallel 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		*	0.9		*
Concrete blocks of compressive strength in N/mm²: 2.8				0.4		0.4
3.5				0.45		0.4
7.0				0.60		0.5
10.5	0.25		0.20	0.75		0.6
14.0 and over				0.90†		0.7†

* *Values not at present available, pending research.*

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

γ_f is obtained from code clause 22, see page 56. If the panel is non-loadbearing and is not designed to resist in-plane wind forces γ_f may be taken as 1.2; if however removal of the panel would impair the stability of the structure γ_f must be taken as 1.4.

The design moment of resistance per unit height

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

where:

f_{kx} is the characteristic flexural strength, obtained from code Table 3, for bending about a vertical axis ie in the column of code Table 3 'Plane of failure perpendicular to the bedjoint'.
γ_m is the partial safety factor for material strength obtained from code Table 4.
Z is the section modulus.

The code does not specifically state that there is no need to check the bending resistance in a two way spanning panel in other than the horizontal direction, because the derivation of the α values given in the code makes allowance for the orthogonal ratio, and checking vertical bending will result in the same answer as checking horizontal bending. This can best be demonstrated algebraically:

horizontal bending, $f_{kx\ horiz} \times Z, = \alpha W_k L^2$ ....(1)

vertical bending, $f_{kx\ vert} \times Z, = \mu\alpha W_k L^2$ ....(2)

but $f_{kx\ vert} = \mu f_{kx\ horiz}$

where $f_{kx\ horiz}$ is the flexural strength in the perpendicular direction and $f_{kx\ vert}$ is the flexural strength in the parallel direction, see Table 10 above Substituting in (2) for $f_{kx\ vert}$ and cancelling results in equation (1).

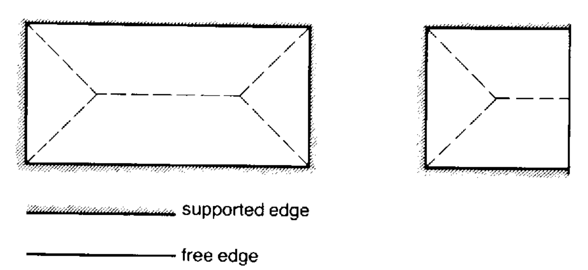
Note that both equations (1) and (2) include the horizontal dimension of the panel. Had they been written in terms of the vertical dimension, the value of α would have altered accordingly.

The code permits any vertical load which acts to increase the flexural strength in the parallel direction to be used to enhance the lateral strength of the panel. The bending moment coefficients in the code are derived on the basis that the ultimate failure pattern in the panel is similar to fracture line failure patterns in reinforced concrete slabs. It is reasonable therefore to assume that the strength will be enhanced only in panels with four sided support, or panels supported top and bottom and on one side, see Figure 55, but not in panels supported at the bottom and two sides, where the critical stress will occur at the top of the panel where there is no vertical load.

The vertical load increases the effective flexural strength in the parallel direction, so modifying the orthogonal ratio of the masonry and thus

Figure 55 Wall panels for which orthogonal ratio may be modified to allow for vertical load.

55



reducing the bending moment coefficient used to calculate the design bending moment. If g_d is the design vertical dead load per unit area, to be taken as $0.9G_k$, the modified orthogonal ratio

$$= \frac{f_{kx\ vert} + \gamma_m g_d}{f_{kx\ horiz}}$$

The need for the γ_m in the dead load term is explained on page 46. If the vertical load is due to the self-weight of the wall itself only the weight of the upper half should be considered.

For laterally loaded walls which span vertically, the code clause 36.4.1 suggests a design bending moment of:

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

which may be modified if the top and bottom support conditions justify the assumption of some end fixity. Earlier in the same clause it is permitted that f_{kx} in the parallel direction may be modified to allow for an increase in flexural strength due to self-weight and any vertical loading in a similar manner to that discussed above for panel walls. The design moment of resistance of the vertically spanning wall becomes

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

where f_{kx} is the characteristic flexural strength in the parallel direction.

This is the same design moment of resistance as given in code clause 36.5.3 for freestanding walls.

The design moment of resistance of cavity walls may be taken as the sum of the design moments of resistance of the two leaves provided that the wall ties used are capable of transmitting the compressive forces to which they are subjected. If the leaves have different orthogonal ratios, the design moments should be calculated assuming that the lateral load is shared between the leaves in proportion to their design moments of resistance. Further explanation of the sharing of loads is given on page 46.

Thus it is first necessary to calculate the design moment of resistance (MR) of each leaf, as on page 65, and then apportion the design load between the leaves as follows:

design load on inner leaf

$$= \gamma_f W_k \frac{\text{M.R. inner leaf}}{\text{M.R. inner leaf} + \text{M.R. outer leaf}}$$

design load on outer leaf $= \gamma_f W_k$—design load on inner leaf.

The design bending moment on each leaf is calculated using the appropriate coefficient α from code Table 9 and compared with the design moments of resistance already calculated.

Method (b)

In certain circumstances, laterally loaded walls may be designed assuming that an arch develops within the thickness of the wall. At the present time, in laterally loaded panels with little vertical load, horizontal arching only may be considered although the code does give a method for designing walls arching vertically under axial load in clause 36.8 which is discussed on page 67. The assumed three-pin arch is shown in Figure 56.

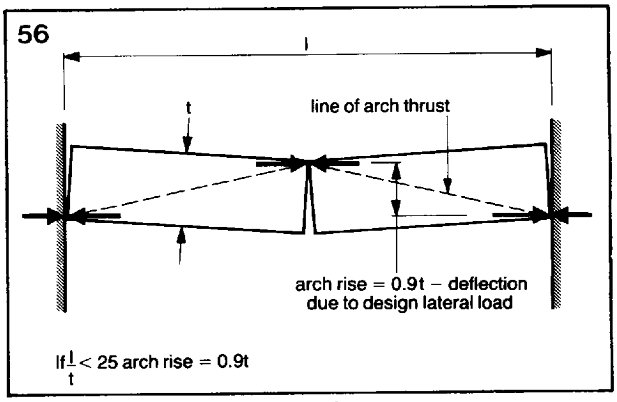


Figure 56 Notional three-pin arch.

If the length to thickness ratio exceeds 25, allowance for the wall deflection must be made in the arch rise as follows:

$$\text{rise} = t - \frac{t}{10} - \text{deflection due to design lateral load.}$$

The maximum design arch thrust per unit height assuming a solid masonry to support junction is:

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

Assuming the wall deflection is small enough to be ignored, the design lateral strength, q_{lat},

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

where:

f_k is the characteristic compressive strength of the masonry, measured in the direction in which the bricks are used (see code clause 23.2).

γ_m is the partial safety factor for materials.

It is, of course, necessary for the supports to be able to withstand the arch thrust.

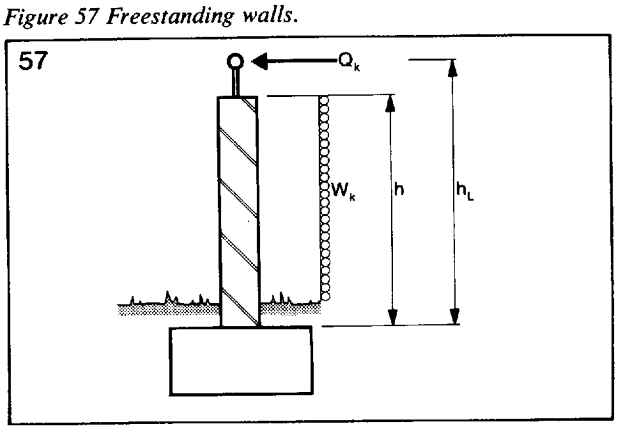
Freestanding walls
(code clause 36.5)

Freestanding walls may be designed either as vertical cantilevers or as panels spanning between piers or other abutments to code clause 36.4. The piers or abutments must then be designed to resist the panel reaction.

If a freestanding cantilever wall is subjected to horizontal forces due to the characteristic wind load, W_k, and a characteristic imposed load, Q_k, as shown in Figure 57, then the design bending moment on the wall is given by:

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

Figure 57 Freestanding walls.



γ_f, the partial safety factor for loads is taken as 1.2 for freestanding walls.
The design moment of resistance is given by:

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

where:
f_{kx} is the characteristic flexural strength
g_d is the design vertical dead load per unit area
Z is the section modulus.

The value of f_{kx} to be used will be that taken from code Table 3 for the plane of failure parallel to the bed joints. If the wall incorporates a damp proof course, which has been proved by test to transmit tension, the value of f_{kx} used will be based on the value at the dpc, but not more than the appropriate masonry value. If the dpc cannot transmit tension, only the self-weight of the wall can be used to resist overturning.

The design vertical dead load must be taken as 0.9 G_k and the section modulus may be calculated to allow for any stiffening effect the wall geometry may give, eg curved or zig-zag in plan.

When the designer cannot rely upon the flexural strength of the wall, eg because of the type of dpc used, the design moment of resistance per unit length

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

where:
t is the wall thickness
n_w is the design vertical load per unit length of wall (taken as 0.9 G_k)
f_k is the characteristic compressive strength of masonry.

This formula is based upon the assumption of a rectangular stress block at the leeward edge of the wall as shown in Figure 34, and consequently the wall is assumed to be cracked. The derivation of the formula is also shown in the figure.

Design lateral strength of axially loaded walls and columns.
(code clause 36.8)

The code permits the designer to deal with the design of axially loaded walls and columns in two ways:

(a) by adjusting the eccentricity of the design vertical load to include the effect of the bending moment due to the lateral load, see Figure 58, and then designing the wall for compressive strength using the capacity reduction factor as usual. When this method is used, the design vertical load is that appropriate to the loading case being considered, usually 0.9 G_k, although γ_f on the wind force may be taken as 1.2 rather than 1.4 if removal of the wall will not impair the stability of the building. In order to keep the resultant eccentricity to a reasonable value (the largest eccentricity for which values of β are given in code Table 7 is 0.3t, although β for eccentricities greater than 0.3t may be calculated in accordance with code Appendix B), a

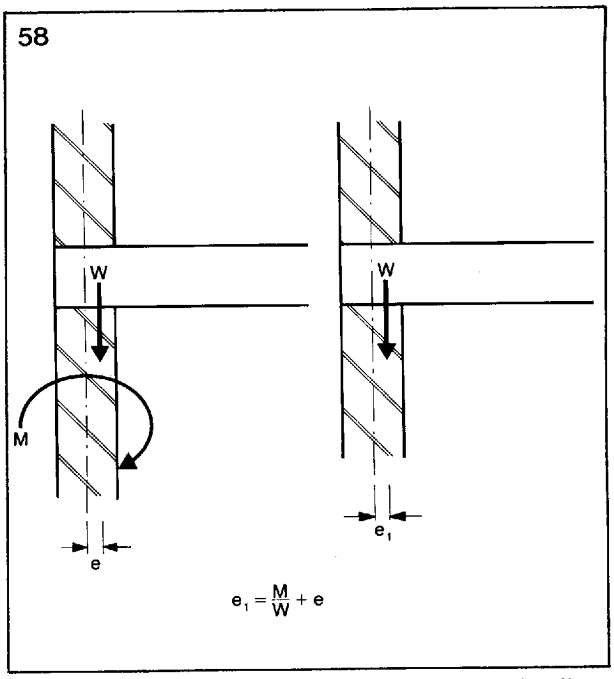


Figure 58 Resultant eccentricity of vertically loaded walls carrying lateral loads.

relatively high design vertical load is required. This limits the application of this method.

$$e_1 = \frac{M}{W} + e$$

where:
M is the design moment due to the wind force
W is the design vertical load
e is the eccentricity of W
e_1 is the resultant eccentricity of W.

(b) by calculating the design lateral strength of the wall or column q_{lat}, assuming the vertical load, n, resists the arch thrust in the wall or column due to the lateral load, using the relationship

$$q_{lat} = \frac{8\,t\,n}{h^2\,\gamma_m}$$

where:
t is the actual thickness of the wall or column
h is the clear height of the wall or column.

To use this approach the ratio $\frac{h}{t}$ must not exceed 25 in the case of narrow brick walls or 20 in other cases.

Normally, n is based on the appropriate design dead load, but when considering the possible effects of misuse or accident it should be taken as 0.95 G_k. The design load must produce a minimum stress of 0.1 N/mm^2.

The form of construction of the floors or members above and below the wall or column must provide adequate lateral support and resistance to rotation of the top and bottom of the member for its full width. This effectively limits the floor construction to reinforced concrete. Any damp proof course etc, in the wall or column must be able to transmit the relevant horizontal forces.

Where the wall or column has return walls, as shown in Figure 59, the value of q_{lat} may be multiplied by the factor k from code Table 10

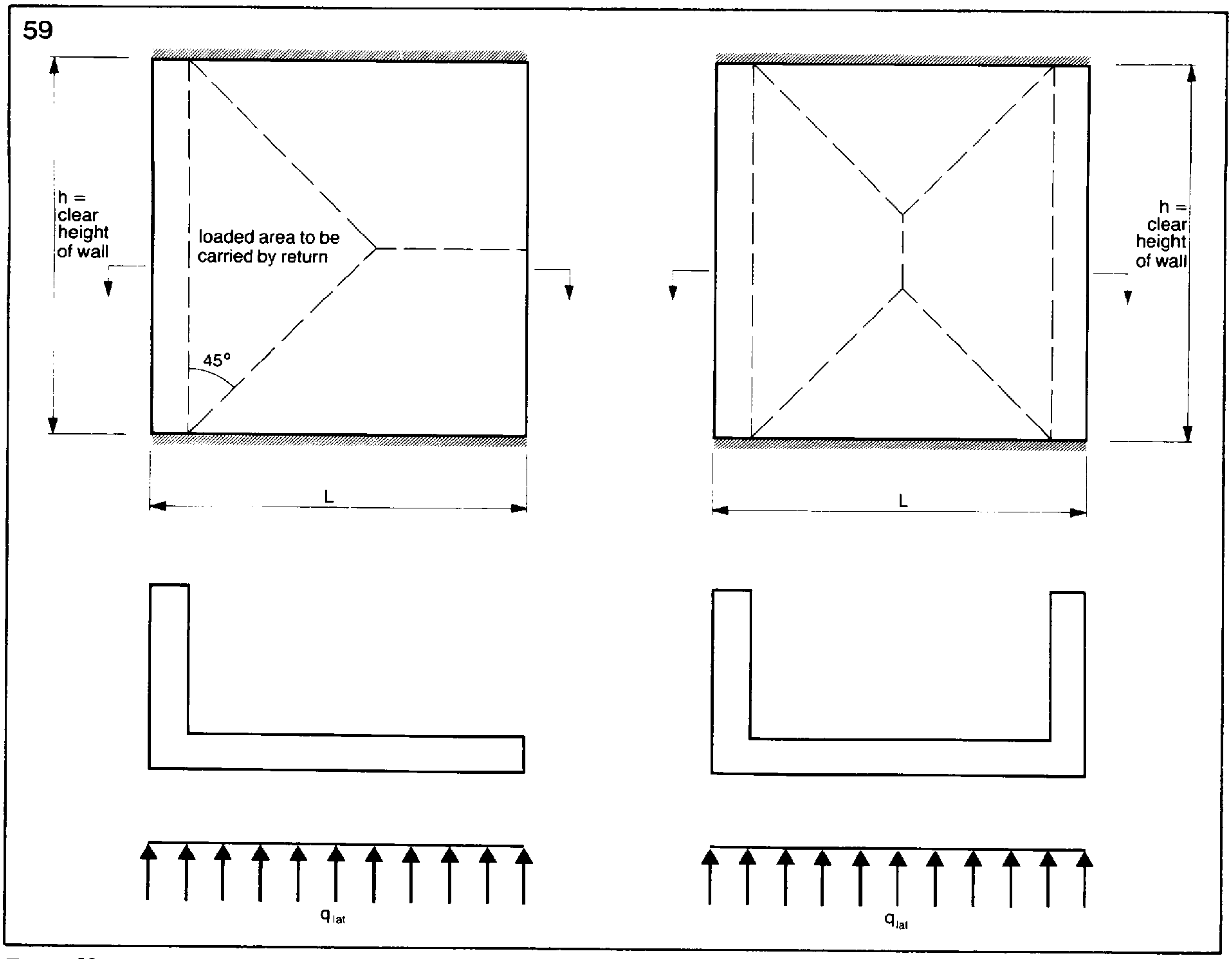


Figure 59 Distribution of lateral loads on wall panel between supports.

reproduced below as Table 11. The returns must be capable of resisting the horizontal reactions transmitted to them.

Table 11 Factor k

Number of returns	Value of k $\frac{L}{h}$ 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

Accidental damage
(code clauses 22 and 32)

Category 2 buildings ie those of 5 storeys and above must, in addition to satisfying the code requirements for Category 1 buildings for stability and robustness, also be designed to cope with the possible effects of accidental damage. The code recommends that this is dealt with by making an assessment of the residual stability and spread of damage following the removal of a loadbearing element, or alternatively that provision is made within the structure for vertical or horizontal tying or both. Three design options are given in the code for satisfying these requirements as follows:

Option 1: All vertical and horizontal loadbearing elements, as defined in code Table 11, must be proved removable, one at a time, without causing collapse, unless they are protected members.

Option 2: Horizontal peripheral, internal and column or wall ties must be provided in accordance with code Table 13, see Figure 39. All vertical elements must be proved removable unless they are protected members.

Option 3: Horizontal ties must be provided as for option 2 and vertical ties must be provided in accordance with code Table 14.

Option 1 is the most satisfactory since the actual behaviour of the building is considered under accidental loading. Load case (d) on page 56 is used in checking that the loadbearing members are removable.

Engineering judgement is required to decide upon the structural effect of the removal of loadbearing elements.

It is necessary only to prove vertical loadbearing elements removable in the top few storeys of a building as such elements will usually be protected below. A protected element is one that can withstand, together with its essential supports, an accidental design load of 34 kN/m^2 applied in any direction. For vertical masonry elements this can be checked using the clause 36.8 equation:

$$q_{lat} = \frac{8\,t\,n}{h^2\,\gamma_m}$$ see page 67.

It should be noted that the axial load per unit length, n, should be calculated for load case (d) page 56 and γ_m should be taken as 1.05.

Option 2 will often be used for loadbearing masonry buildings since horizontal ties can be accommodated easily in the reinforced concrete slabs normally used in masonry structures of 5 storeys and over. The design procedure for proving the vertical loadbearing elements removable or protected is the same as for option 1.

Option 3 is unlikely to be used often for loadbearing masonry buildings since the vertical tie requirement involves the inclusion of continuous ties from roof level to a level at which the relevant members are protected or to foundation level, and the simplest way of providing these is as reinforced concrete columns. Also, the minimum thickness of a solid wall or the loadbearing leaf of a cavity wall is 150mm. These constraints will not usually be acceptable in loadbearing masonry structures.

In options 2 and 3, peripheral horizontal ties must be designed to resist a basic horizontal force, F_t, equal to 60 kN or $20+4N_s$ whichever is less, where N_s is the number of storeys including ground floor and basement. Internal ties must be designed for a force of F_t or

$$\frac{F_t(G_k+Q_k)}{7.5} \times \frac{L_a}{5}$$

whichever is greater, where L_a is the lesser of the distance between the centres of the vertical loadbearing members, in the direction of the tie, or 5 times the clear storey height. If the tie being considered is perpendicular to the span of a one-way spanning slab, it need only be designed to resist F_t. External column or external wall tie connections must be designed to resist $2F_t$, or F_t times the clear storey height divided by 2.5.

Vertical ties must be designed for a force of:

$$\frac{34A}{8000}\left(\frac{h_a}{t}\right)^2 \text{ N or 100 kN/m length of wall or}$$

column whichever is the greater.
where:
A is the horizontal cross-section area in mm² of the column or wall, including piers, or the loadbearing leaf of an external cavity wall.
h_a is the clear height of a column or wall.
t is the thickness of the column or wall.

The required locations of these ties are shown diagrammatically in Figure 39. The exact locations and fixing requirements are given in code Tables 13 and 14.

DESIGN EXAMPLES

4

The following chapter contains two worked examples which attempt to cover as many as possible of the design aspects of BS 5628 Part 1. Where different forms of construction might be used or where different methods of analysis could be applied, examples of such design have been incorporated by the device of parallel calculations. The parallel calculations are printed in colour so that they may be distinguished easily from the main example calculations.

The examples are cross referenced in the left-hand margin to the relevant clause numbers in the code of practice. Reference is also made to Chapter 3 of this handbook.

The first example is for the design of a multi-storey loadbearing brick structure. The building is seven storeys high and has cavity brickwork external walls and one brick thick internal structural walls. The roof and floor slabs are in reinforced concrete. The layout of the structural walls falls somewhere between a crosswall arrangement with substantial return walls, and a fully cellular arrangement and repeats at each level so ensuring a stable and robust structure. Overturning forces on the building are resisted by the brick walls. The example illustrates design for stability, design for axial and eccentric compressive loads, design lateral strength of axially loaded walls and design for accidental damage.

The second example is for the design of a three storey, end of terrace, house in structural masonry. The houses of the remainder of the terrace are only two storeys in height. The house has a trussed rafter main roof and a jack rafter pitched roof where the building steps back at second floor level. The front external wall to the second floor is carried on a reinforced concrete beam which is supported on one leaf of the party cavity wall, an internal loadbearing wall and the inner leaf of the external cavity wall. This building illustrates the design of structural blockwork, design for concentrated loads and the design of laterally loaded wall panels.

In both examples the partial safety factor for material strength, γ_m, has been taken generally as 3.5 ie assuming normal category of both manufacturing and construction control. The effect of varying the value of γ_m is shown in the parallel calculations.

Two further example calculations which could not sensibly be a part of Examples 1 and 2 are given at the end of the chapter for the design of masonry walls stiffened by piers and for laterally loaded masonry infill panels to framed structures.

EXAMPLE 1
Seven-storey loadbearing brickwork block of flats

A typical plan of and cross-section through the building are shown in figure 60.

The external cavity walls are 280 mm thick with a 25 mm insulating lining within the cavity. Both leaves are of brickwork.

The internal loadbearing walls are 215 mm brickwork.

The floors and roof are 150 mm reinforced concrete slabs spanning between the internal and external walls and bearing on the inner leaf of the external cavity walls. The floor
29.2 ***slab is extended outwards at the third and sixth floor levels to support the outer leaf, and the external walls are restrained at ground floor level.***

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The storey height is 2600 mm.

The building is located on the outskirts of Bristol.

Loading

21. ***Characteristic loads***

Roof load:

Dead loads, G_k,	75 mm lightweight screed	1.0 kN/m²
	150 mm rc slab	3.6 kN/m²
	felt, chippings, services, etc	0.4 kN/m²
		5.0 kN/m²
Imposed load, Q_k,		1.5 kN/m²

Floor load:

Dead load, G_k,	50 mm screed	1.2 kN/m²
	150 mm rc slab	3.6 kN/m²
	partitions	2.0 kN/m²
	services	0.1 kN/m²
		6.9 kN/m²
Imposed load, Q_k,		2.0 kN/m²

Self-weight of walls, G_k:

External walls	
102.5 mm outer leaf	2.25 kN/m²
102.5 mm inner leaf, plastered one side	2.50 kN/m²
280 mm cavity walls, plastered one side	4.75 kN/m²
Internal walls	
215 mm structural walls plastered both sides	5.0 kN/m²

The value for the live load, Q_k, on the floors is taken here as 2.0 kN/m², a figure referred to in the Building Regulations. It is more usual for this load to be taken as 1.5 kN/m², as given in CP 3: Chapter V: Part 1: 1967. Whatever value is used, the design method remains the same.

Wind loading (from CP 3: Chapter V: Part 2: 1972)

Assumed basic wind speed $=43$ m/s

Wind speed factors:

$S_1 = 1.0$

$S_2 = 0.88$ using ground roughness category (3) class B, height of building $=18.2$ m.

$S_3 = 1.0$

Design wind speed, V_s, $= 43.0 \times 1.0 \times 0.88 \times 1.0 = 37.8$ m/s

Therefore, dynamic wind pressure, $q, = \dfrac{0.613 \times 37.8^2}{10^3} = 0.875$ kN/m²

Characteristic vertical loads on walls

Only the most heavily loaded walls will be designed. These walls are numbered 1, 2, 3 and 4 in Figure 60.

The characteristic loads on the walls from each floor and the roof are as follows:

Wall 1 (inner leaf only)

Span of slab $\div 2 = 1.83$ m

Load from roof: dead load, G_k, $= 5.0 \times 1.83 = 9.2$ kN/m

imposed load, Q_k, $= 1.5 \times 1.83 = 2.7$ kN/m

Load from one floor: dead load, G_k, $= 6.9 \times 1.83 = 12.6$ kN/m

imposed load, Q_k, $= 2.0 \times 1.83 = 3.7$ kN/m

Wall self-weight per storey height $= 2.5 \times 2.45 = 6.1$ kN/m

Wall 2 (inner leaf only)

Length of wall $= 1.2$ m, width of slab carried $= 2.25$ m, span of slab $\div 2$ as wall 1.

Load from roof: dead load, G_k, $= 5.0 \times 1.83 \times \dfrac{2.25}{1.2} = 17.2$ kN/m

imposed load, Q_k, $= 1.5 \times 1.83 \times \dfrac{2.25}{1.2} = 5.2$ kN/m

Load from one floor: dead load, G_k, $= 6.9 \times 1.83 \times \dfrac{2.25}{1.2} = 23.7$ kN/m

imposed load, Q_k, $= 2.0 \times 1.83 \times \dfrac{2.25}{1.2} = 6.9$ kN/m

Wall self-weight per storey height, inner leaf only $= 2.5 \times 2.45 = 6.1$ kN/m

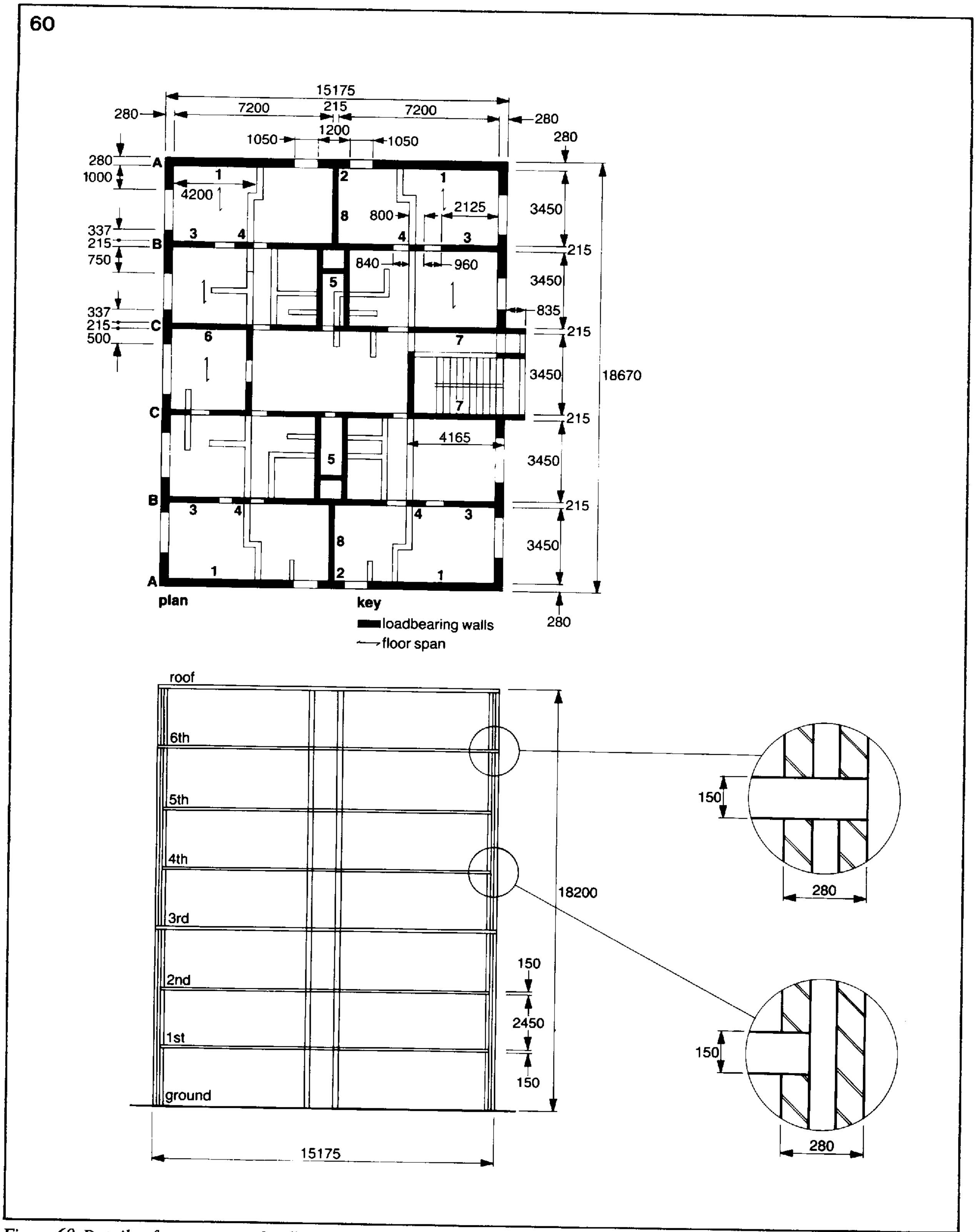


Figure 60 Details of seven storey loadbearing masonry building.

Wall 3

Length of wall $=2.125$ m, width of slab carried $=2.545$ m, total span of slab $\div 2 = 3.665$ m

Load from roof: dead load, G_k, $=5.0 \times 3.665 \times \dfrac{2.545}{2.125} = 22.0$ kN/m

imposed load, Q_k, $=1.5 \times 3.665 \times \dfrac{2.545}{2.125} = 6.6$ kN/m

Load from one floor: dead load, G_k, $=6.9 \times 3.665 \times \dfrac{2.545}{2.125} = 30.3$ kN/m

imposed load, Q_k, $=2.0 \times 3.665 \times \dfrac{2.545}{2.125} = 8.8$ kN/m

Wall self-weight per storey height $=5.0 \times 2.45$ $=12.3$ kN/m

Wall 4 (total load)
Width of slab carried $=1.7$ m, total span of slab $\div 2$ as for wall 3.
Load from roof: dead load, G_k, $=5.0\times3.665\times1.7=31.2$ kN
imposed load, Q_k, $=1.5\times3.665\times1.7=9.4$ kN
Load from one floor: dead load, G_k, $=6.9\times3.665\times1.7=43.0$ kN
imposed load, Q_k, $=2.0\times3.665\times1.7=12.5$ kN
Wall self-weight per storey height $=5.0\times2.45\times0.8=9.8$ kN

Total characteristic loads at each level are tabulated in Table 12, the imposed loads have been reduced in accordance with CP3: Ch. V: Part 1: 1967 – Table 2.

Walls 1 to 4 will first be checked to ensure that the part of the code requirements for overall stability relating to resistance to horizontal forces is satisfied. Stresses derived from this analysis will then be used to design these walls for compressive strength.

Table 12 Characteristic loads on walls (kN/m)

	Wall 1*		Wall 2*		Wall 3		Wall 4	
Level of loading	G_k	Q_k	G_k	Q_k	G_k	Q_k	G_k	Q_k
underside of roof slab	9.2	2.7	17.2	5.2	22.0	6.6	31.2	9.4
underside of 6th floor slab	27.9	5.8	47.0	10.9	64.6	13.9	84.0	19.7
underside of 5th floor slab	46.6	8.1	76.8	15.2	107.2	19.4	136.8	27.5
underside of 4th floor slab	65.3	9.7	106.6	18.1	149.8	23.1	189.6	32.8
underside of 3rd floor slab	84.0	10.5	136.4	19.7	192.4	25.1	242.4	35.7
underside of 2nd floor slab	102.7	12.7	166.2	23.8	235.0	30.4	295.2	43.2
underside of 1st floor slab	121.4	15.0	196.0	27.9	277.6	35.7	348.0	50.7
just above ground floor slab	127.5	15.0	202.1	27.9	289.9	35.7	357.8	50.7

**The tabulated characteristic loads for walls 1 and 2 assume the inner leaf of the wall is load bearing and include the self-weight of the inner leaf only. The outer leaf is assumed to carry its own weight.*

The characteristic loads tabulated in Table 12 are the loads at the underside of each roof or floor slab (except for that just above the ground floor slab). Thus the characteristic dead load at the underside of the 4th floor slab of wall 3 is as follows:

roof load G_k = $= 22.0$ kN/m
4th, 5th and 6th floor loads $G_k = 3\times30.3$ $= 90.9$ kN/m
self-weight of wall (3 storeys) $=3\times12.3$ $= 36.9$ kN/m

characteristic dead load at underside of 4th floor $=149.8$ kN/m

Similarly the characteristic imposed load at underside of the 4th floor slab of wall 3:

roof load Q_k $= 6.6$ kN/m
4th, 5th and 6th floor loads Q_k $= 26.4$ kN/m

33.0 kN/m
30% imposed load reduction $= -9.9$ kN/m

characteristic imposed load at underside of 4th floor $= 23.1$ kN/m

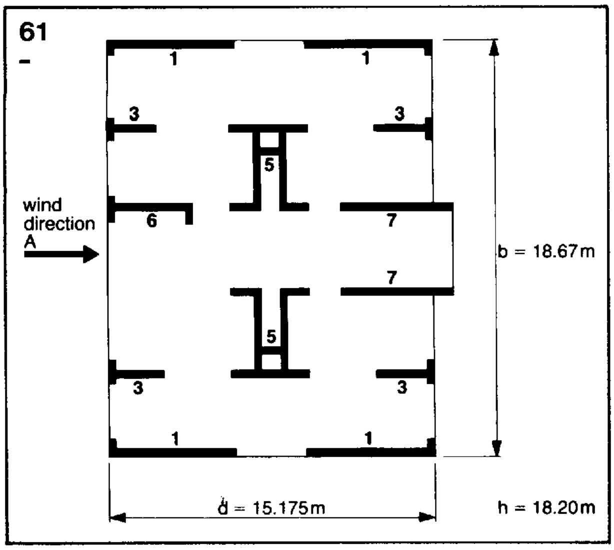
20. **Stability**

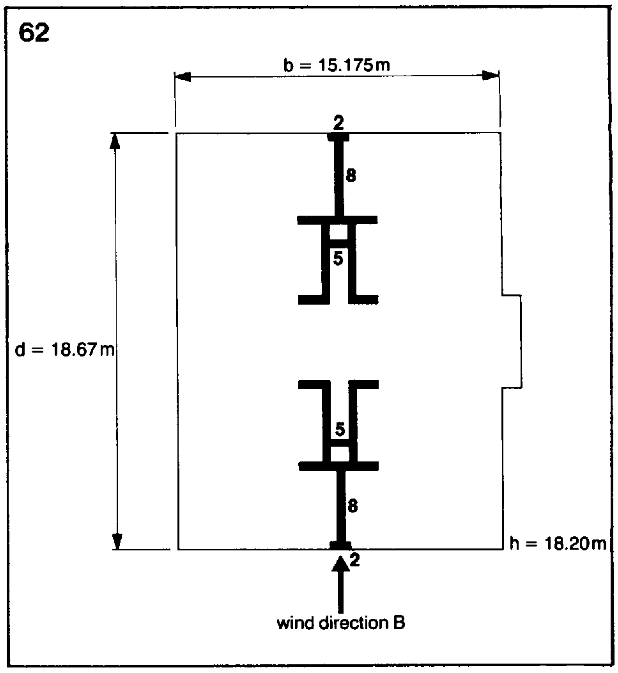
The structure must be capable of resisting horizontal loads due to:

1. Characteristic wind pressure
2. 1.5% of the total characteristic dead load above any level considered as a uniformly distributed horizontal load
3. Accidental forces

3.4 *Characteristic wind loading (CP 3: Chapter V: Part 2, Table 10)*

Force coefficients, C_f:
In direction A (Figure 61):





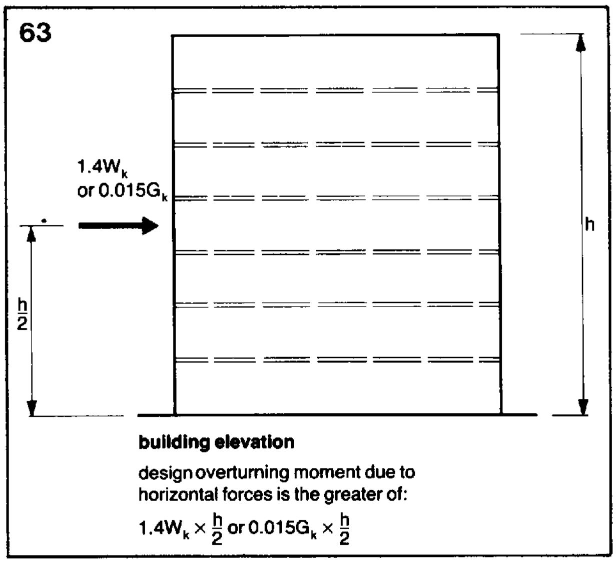


Figure 61 Walls resisting horizontal forces in direction 'A'.

Figure 62 Walls resisting horizontal forces in direction 'B'.

Figure 63 Overall stability – overturning due to horizontal forces.

$$\frac{l}{w} = \frac{18.67}{15.175} = 1.23$$

$$\frac{b}{d} = \frac{18.67}{15.175} = 1.23$$

$$\frac{h}{b} = \frac{18.20}{18.67} = 0.97$$

Therefore $C_f = 1.0$

The dynamic wind pressure, $q_s = 0.875$ kN/m², see page 72.

Therefore the total characteristic wind load in direction A,

$W_k = 0.875 \times 18.2 \times 1.0 \times 18.67$

$= 298$ kN

In direction B (Figure 62):

$$\frac{l}{w} = 1.23$$

$$\frac{b}{d} = \frac{15.175}{18.67} = 0.81$$

$$\frac{h}{b} = \frac{18.2}{15.175} = 1.2$$

Therefore $C_f = 0.95$

The total characteristic wind load in direction B,

$W_k = 0.875 \times 15.175 \times 0.95 \times 18.2$

$= 230$ kN

Characteristic dead load, G_k, of whole building at ground level

Total plan length of external wall = 47.5 m

Total plan length of internal wall = 78.0 m

Walls:	$7 \times 4.75 \times 2.45 \times 47.5$	= 3870 kN
	$7 \times 5.0 \times 2.45 \times 78.0$	= 6700 kN
Roof:	$5.0 \times 15.175 \times 18.670$	= 1410 kN
Floors:	$6 \times 6.9 \times 15.175 \times 18.670$	= 11700 kN
		23680 kN

Design loading

22(b) Design horizontal load is the greater of 1.4 W_k or 0.015 G_k.

$0.015\ G_k = 0.015 \times 23680 \quad = 355$ kN

In direction A, $1.4\ W_k = 1.4 \times 298 \quad = 417$ kN

Hence, design horizontal load is based on wind loading.

In direction B, $1.4\ W_k = 1.4 \times 230 \quad = 322$ kN

Hence, design horizontal load is 0.015 G_k.

Direction A

Having established that the design horizontal load is 1.4 W_k, it is convenient to revert to characteristic loading at this stage. The characteristic overturning moment due to wind loading, see Figure 63

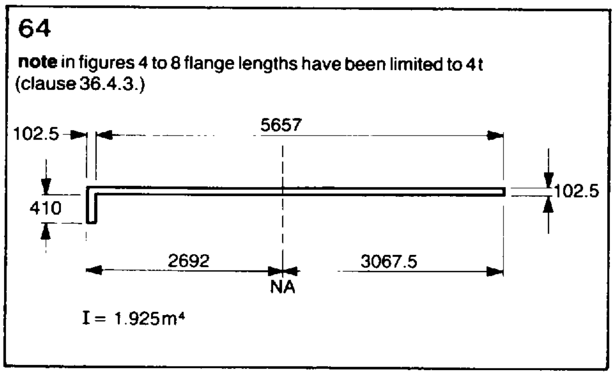
$$= 298 \times \frac{18.2}{2} = 2720 \text{ kN/m.}$$

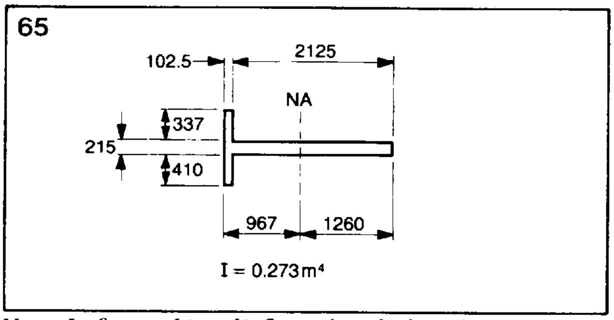
Following normal loadbearing design procedure, this moment is shared between the walls shown in Figures 64 to 68 in proportion to their stiffness, ie, their moments of inertia, as all the walls have the same height.

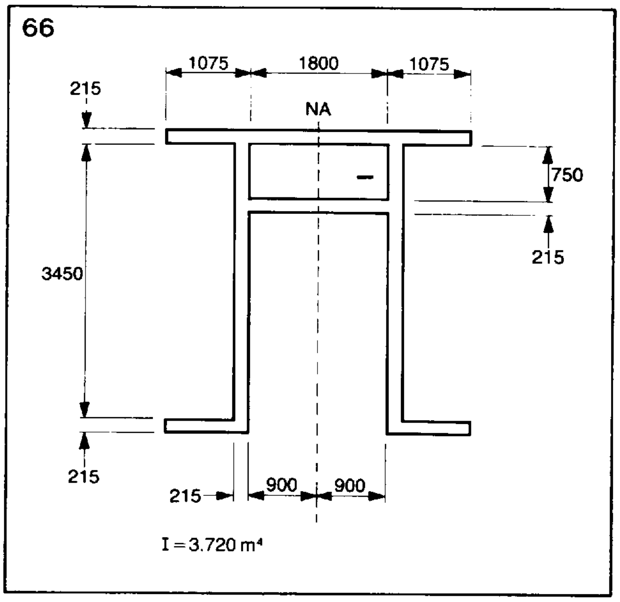
The characteristic shear force and bending moment on each wall are tabulated in Table 13.

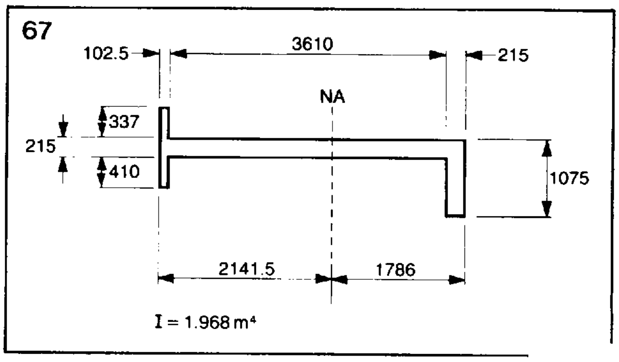
Table 13 Bending moments and shear forces due to characteristic loading in direction 'A'.

Wall no.	No. off	Moment of inertia, I_w, of each wall (m^4)	Total moment of inertia, I_t (m^4)	$\frac{I_w}{\Sigma I_t}$	Shear force V_w on each wall (kN)	Total shear force V_t (kN)	B. moment M_w on each wall (kNm)	Total B. moment M_t (kNm)
1	4	1.925	7.700	0.0849	25.3	101	230.9	924
3	4	0.273	1.092	0.0120	3.6	14	32.6	131
5	2	3.720	7.440	0.1640	48.9	98	446.0	892
6	1	1.968	1.968	0.0868	25.9	26	236.1	236
7	2	2.240	4.480	0.0988	29.4	59	268.7	537
Summation			22.680	—	—	298	—	2720









Note: In figures 64 to 68 flange lengths have been limited to 4t (clause 36.4.3)

Figure 64 Wall 1

Figure 65 Wall 3

Figure 66 Wall 5

Figure 67 Wall 6

Table 14 Stresses at bottom of walls due to characteristic loading in direction 'A'.

Wall no.	Stress at extreme fibre* (N/mm²) y minimum	y maximum
1	±0.37	±0.32
3	±0.15	±0.12
5	±0.24	±0.24
6	±0.26	±0.21
7	±0.30	±0.30

* *stress* $= \pm \frac{M_w}{I_w} y$

where M_w and I_w are obtained from Table 13 and y minimum and y maximum are obtained from Figures 64 to 68 (distances from neutral axis to extreme fibre).

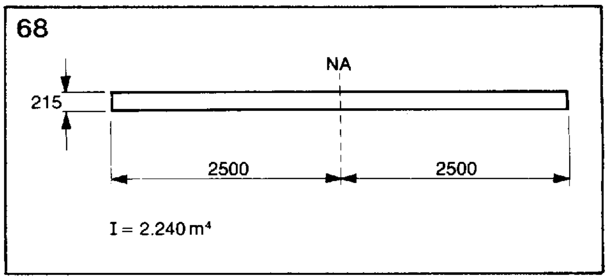


Figure 68 Wall 7

22(b) *Stresses due to design vertical and horizontal loads*
The load case to be considered is Dead and Wind load so that the partial safety factor, γ_f, $=0.9$ on dead loads and 1.4 on wind loads.

Stress due to design self-weight of wall only $= \frac{0.9 \times 5.0 \times 2.6 \times 7}{1.0 \times 215} = 0.38$ N/mm²

Stress due to design wind loading ($\gamma_f \times$ stress from Table 14):
Wall 1 $= 1.4 \times 0.37 = 0.52$ N/mm²
Wall 7 $= 1.4 \times 0.30 = 0.42$ N/mm²
By inspection the stress due to design wind loading in the other walls is exceeded by the design self-weight stress.

It can be seen that only a small amount of dead load from the floor slabs on these walls will prevent any residual tension occurring in walls 1 and 7.
Design dead load from roof and floor slabs required to prevent residual tension in wall 1

$$= \frac{(0.52\text{–}0.38)\ 102.5 \times 1000}{1000} = 14.4 \text{ kN/m}$$

Now, from page 72, characteristic dead load from roof slab and one floor slab $= 9.2 + 12.6 = 21.8$ kN/m
Therefore design dead load $= 0.9 \times 21.8 = 19.6$ kN.
This exceeds 14.4 kN/m and there will therefore be no residual tension under wall 1.

Design dead load from roof and floor slab required to prevent residual tension in wall 7

$$= \frac{(0.42\text{–}0.38)\ 102.5 \times 1000}{1000} = 4.1 \text{ kN/m}$$

From page 72 characteristic dead load of roof $= 5.0$ kN/m²
Therefore design dead load $= 0.9 \times 5.0 = 4.5$ kN/m² and only 1.0 m width of roof slab is required to prevent residual tension. Actual width of roof slab carried by wall 7 is 3.45 m.

33. *Shear – direction A*
30. The shear force is distributed between the walls in proportion to their stiffness in Table 13. As only walls 1 to 4 in Figure 60 will be fully designed, walls 5, 6 and 7 will be checked here as follows:

The plan area of each wall to be considered as resisting shear is that area of the wall that forms the web of the section. The shear stress in each wall is given in Table 15, see Chapter 3 page 63.

Table 15 Shear stress in walls 5, 6 & 7 due to design loads.

Wall no.	Characteristic shear force, V, (kN)	Area of wall resisting shear, A, (m²)	Shear stress due to design loads v_h, * (N/mm²)
5	48.9	1.79	0.038
6	25.9	0.84	0.043
7	29.4	1.07	0.038

$*v_h = \frac{1.4V}{A}$

33. Design shear strength of masonry $= \frac{f_v}{\gamma_{mv}}$
25. where characteristic shear strength $f_v = 0.35 + 0.6\, g_A$
27.4 and $\gamma_{mv} = 2.5$

The constant component of the design shear strength

$=\frac{0.35}{2.5}=0.14$ N/mm²

This exceeds the stresses due to design loads in Table 15 and is therefore satisfactory. The increase in shear strength from dead load is not needed.

Direction B (see page 75)

The design overturning moment, Figure 63

$=355\times\frac{18.2}{2}=3230$ kNm.

This moment is shared equally between the two wall complexes made up of walls 5, 8 and 2, as shown in Figure 69.

The design bending moment per wall complex

$=3230\times0.5=1615$ kNm.

Therefore, stresses due to this design bending moment

$=\pm\frac{My}{I}$, as for direction A, page 77.

$=\pm\frac{1615\times4.415}{15.317\times10^3}$ or $\pm\frac{1615\times3.017}{15.317\times10^3}$

$=\pm0.47$ N/mm² or ±0.32 N/mm²

Stress due to design vertical and horizontal loads

The load case to be considered is dead and wind load when the partial safety factor, γ_f, $=0.9$ on the dead load.

Therefore, stress due to design self-weight of wall only

$=0.38$ N/mm² as for direction A, page 77.

It can be seen that the self-weight stress will only be exceeded near the wall 2 end of the wall complex. Using the same method as shown for direction A on page 79, it can be shown that the dead load from the floors is more than sufficient to prevent any residual tension.

Shear – direction B

33. The horizontal design shear force per wall complex is:

$\frac{355}{2}=178$ kN.

This force is resisted by the walls in whose plane it acts, ie, the walls parallel to direction B in Figure 69.

Design shear stress, v_h, $=\frac{178\times10^3}{215(3767+2\times3880)}$

$=0.07$ N/mm²

69

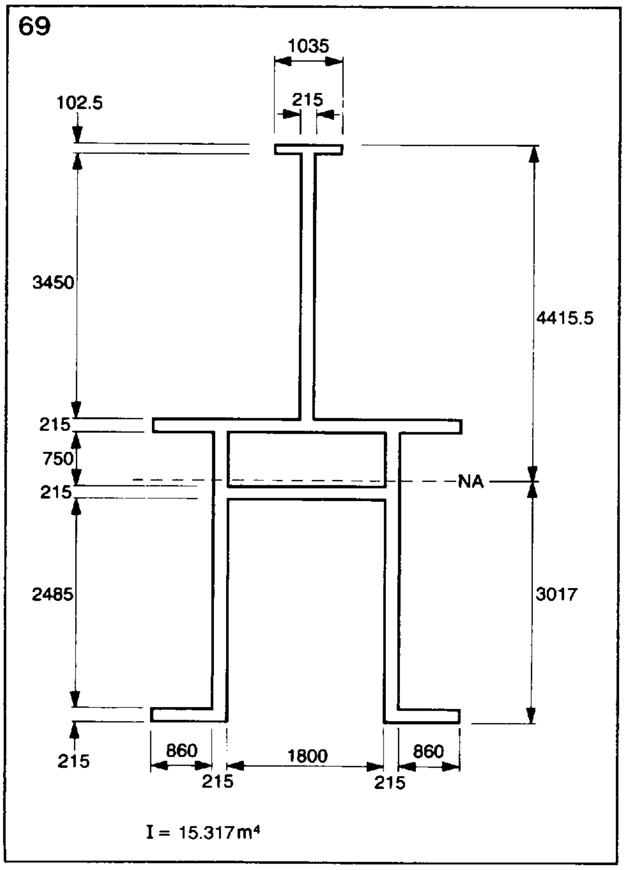


Figure 69 Wall complex made up of walls 2, 5 and 8, see Figure 60 (2 no.)

Therefore minimum characteristic shear strength required
$=\gamma_{mv}\, v_h$
$=2.5 \times 0.07 = 0.175$ N/mm²

25. The characteristic shear strength of masonry, f_v,
$=0.35 + 0.6\, g_A$ N/mm²
exceeds the required characteristic shear strength even without the self-weight component $0.6 g_A$ and is therefore satisfactory.

Characteristic compressive strength of brick required at ground floor of walls 1 to 4, see Figure 60

The critical section of a wall for design for compressive strength is 0.4 h below the level of application of the load, where h is the storey height. For simplicity, the design loads in this example include the full self-weight of the storey height of wall under consideration, rather, than just 0.4 of it.

Wall 1

Characteristic dead load, G_k, $=127.5$ kN/m, from Table 12.
Characteristic imposed load, Q_k, $=15.0$ kN/m, from Table 12.
Characteristic wind stress $=\pm 0.37$ N/mm² or ± 0.32 N/mm², from Table 14.

22. *Design loads*
Consider 1 m length of wall.

22(a) Load case (a), dead and imposed load:
Design dead load $=\gamma_f G_k = 1.4 \times 127.5 = 179$ kN/m
Design imposed load $=\gamma_f Q_k = 1.6 \times 15.0 = 24$ kN/m
Therefore, design vertical load $=203$ kN/m

22(b) Load case (b), dead and wind load:
Design dead load $=\gamma_f G_k = 1.4 \times 127.5 = 179$ kN/m
Design wind load $=\gamma_f W_k = 1.4 \times 0.37 \times 102.5 \times \frac{10^3}{10^3} = 53$ kN/m
Therefore, design vertical load $=232$ kN/m

22(c) Load case (c), dead, imposed and wind load:
Design dead load $=\gamma_f G_k = 1.2 \times 127.5 = 153$ kN/m
Design imposed load $=\gamma_f Q_k = 1.2 \times 15.0 = 18$ kN/m
Design wind load $=\gamma_f W_k = 1.2 \times 0.37 \times 102.5 \times \frac{10^3}{10^3} = 45$ kN/m
Therefore, design vertical load $=216$ kN/m

28. ***Slenderness considerations***
28.2.2 Enhanced lateral resistance is provided by the in situ concrete slab bearing on the inner leaf of the wall.
28.3.1 Effective height, h_{ef}, $= 0.75 \times 2450 = 1840$ mm
28.4 Effective thickness, t_{ef}, $= 0.67\,(2 \times 102.5) = 137$ mm
28.1 Slenderness ratio $= \frac{h_{ef}}{t_{ef}} = \frac{1840}{137} = 13.4$

31. ***Eccentricity at right angles to the wall (see Figure 70)***
Loading (See page 72)
Design loads:

W_1: $1.4\, G_{k1} = 1.4 \times 12.6 = 17.6$ kN/m
$1.6\, Q_{k1} = 1.6 \times 3.7 = 5.9$ kN/m
23.5 kN/m

W_2: $1.4\, G_{k2} = 1.4\,(121.4 - 12.6) = 152.0$ kN/m
$1.6\, Q_{k2} = 1.6\,(15.0 - 3.7) = 18.0$ kN/m
170.0 kN/m

Taking moments about centre line of inner leaf:

$$(170.0 + 23.5)\, e_x = 23.5 \times \frac{t}{6}$$

$$\text{Therefore, } e_x = \frac{23.5 \times t}{6 \times 193.5}$$

$= 0.02\, t$

Hence, from code Table 7, capacity reduction factor, β, $= 0.9$

A larger eccentricity could, perhaps, be obtained by considering the loading case:
$W_2 = 0.9\, G_{k1} + 1.6\, Q_{k1}$
$W_1 = 1.4\, G_{k2} + 1.6\, Q_{k2}$
However, virtually always, the effect of the decrease in the vertical load outweighs the effect of any increase in eccentricity on the design stress, as shown in the chapter on design, and, therefore, this latter loading case need not usually be considered, but see page 57.

32.2.1 ***Design vertical resistance of wall 1***

23.1.2 As the thickness of the inner leaf of wall 1 is equal to the width of a standard format brick, the value of f_k may be multiplied by 1.15.

Therefore, the design vertical resistance of wall 1

$$=\frac{\beta t\,(1.15\,f_k)}{\gamma_m}=\frac{0.9\times102.5\times1.15\,f_k}{3.5}=30.3\,f_k$$

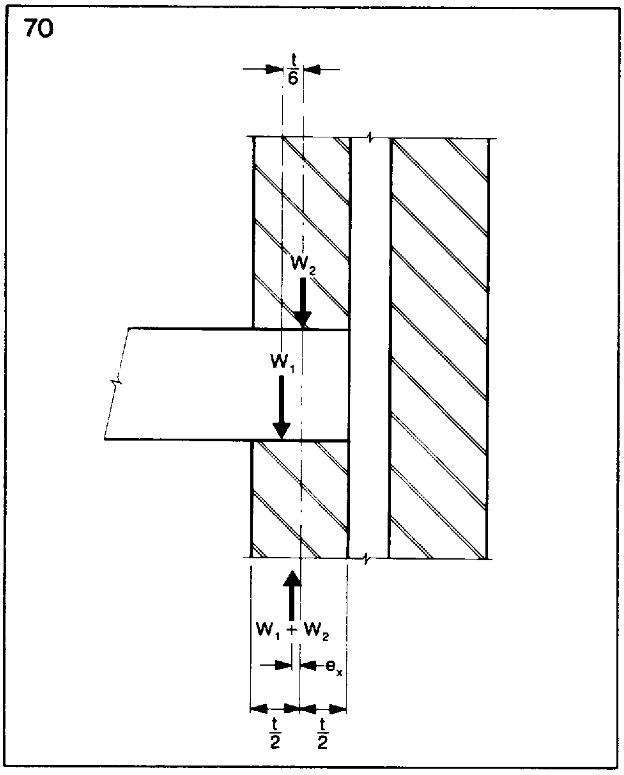


Figure 70 Eccentricity at right angles to wall 1.

27.3 The partial safety factor for material strength, γ_m, is taken as 3.5 from code Table 4, as both the category of manufacturing and construction control are assumed in this case to be normal.

Design vertical load = 232 kN/m (load case (b)).
Therefore, minimum characteristic compressive strength, f_k, required

$$=\frac{232\times10^3}{30.3\times10^3}=7.7\ \text{N/mm}^2$$

From code Figure 1, class 4 bricks in mortar designation (ii), or class 5 bricks in mortar designation (iii) may be used.

Parallel calculation
If it is possible to obtain both special control of manufacture of the masonry units and of construction, a considerable reduction in the strength of the unit required can be made by the modification of γ_m from 3.5 to 2.5:

32.2.1 ***Design vertical resistance of wall 1***

$$=\frac{0.9\times102.5\times1.15f_k}{2.5}=42.4\,f_k$$

Design vertical load = 232 kN/m
Therefore, f_k required

$$=\frac{232\times10^3}{42.4\times10^3}=5.5\ \text{N/mm}^2$$

Thus, from code Table 1, a class 3 brick in mortar designation (iii) may be used, instead of the class 4 brick required for $\gamma_m=3.5$

33. ***Horizontal shear force in plane of wall 1***
Characteristic shear load due to wind on wall
= 25.5 kN, from Table 13.
Characteristic dead load
= 127.5 kN, from Table 12.
Loading case (b) gives the most severe conditions, ie, no imposed vertical load.
Design wind load = $1.4\times W_k$
Design dead load = $0.9\times G_k$

Therefore, design shear stress, v_h,

$$=\frac{1.4\times25.5\times10^3}{5759\times102.5}=0.061\ \text{N/mm}^2$$

25. Characteristic shear strength of masonry, f_v,
$=0.35+0.6\ g_A$ for walls in mortar designation (i), (ii) or (iii).
Design vertical load per unit area, g_A,

$$=\frac{0.9\times127.5\times10^3}{102.5\times1000}=1.12\ \text{N/mm}^2$$

Therefore, $f_v=0.35+0.6\times1.12$
$=1.02\ \text{N/mm}^2$

27.4 The partial safety factor for shear loads, γ_{mv}, $=2.5$*.
33. Therefore, the design shear resistance of the wall

$$=\frac{f_v}{\gamma_{mv}}=\frac{1.02}{2.5}=0.4\ \text{N/mm}^2$$

This exceeds the design shear stress, v_h, and is therefore satisfactory.
**No alternative values for* γ_{mv} *are given in the code, except when considering the effects of accidental damage.*

Parallel calculation

31. The code states that the eccentricity of loading on a wall should preferably be calculated. The following calculation illustrates one method of doing this. It also illustrates some of the assumptions made by the code in its treatment of eccentricity (see code Appendix B and chapter 2 page 36 . This calculation also considers the case where both leaves of the cavity wall are loadbearing.

The calculation procedure if only the inner leaf is loaded, is similar except that the relative stiffness of the masonry is based on two thirds of the sum of the stiffnesses of the two leafs or the stiffness of the inner leaf alone, whichever is greater, see chapter 3, page 60.

Assume both leaves of the external wall are loadbearing (ie, floor and roof slabs bear fully on both leaves) at all floors.
Consider wall 1:

Design loads – ground floor level

22. The critical load case is, as before, load case (b): dead and wind, see page 79.

32.2.3 In order to apportion the total vertical load between the two leaves of the wall, it is necessary to determine the eccentricity of loading at first floor level. One method of calculating the eccentricity is by the moment distribution method, assuming rigid joints between the masonry and the floor slab, see Figure 71.

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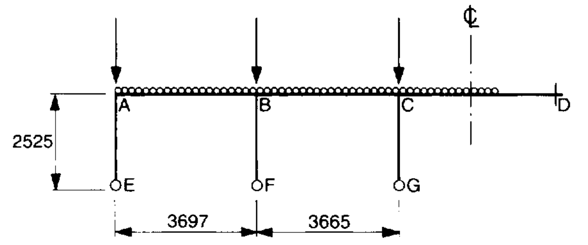

Figure 71 Idealised frame for calculation of eccentricity.

Design dead load $=1.4\times6.9$
$=9.7\ \text{kN/m}^2$

Fixed end moments

Span AB $FM_{AB}=-\frac{wL^2}{12}=-\frac{9.7\times3.697^2}{12}$
$=-11.1\ \text{kNm/m}$

$FM_{BA}=-FM_{AB}$
$=+11.1\ \text{kNm/m}$

Similarly, span BC & CD

$FM_{BC}=FM_{CD}=-\frac{9.7\times3.665^2}{12}$
$=-10.8\ \text{kNm/m}$

$FM_{CB}=FM_{DC}=-FM_{BC}$
$=+10.8\ \text{kNm/m}$

Stiffnesses

Assume:

Young's modulus for concrete $= 25{,}000\ \text{N/mm}^2$

Young's modulus for brickwork $= 8{,}750\ \text{N/mm}^2$ (from modular ratios given in CP 111, Part 2, 1970 for 27.5-34.5 MN/m^2 bricks)

$$\text{Relative stiffness, } k_{AB} = k_{BC} = \frac{EI}{L} = 25{,}000 \times \frac{1000 \times 150^3}{12} \times \frac{1}{3665}$$
$$= 1.92 \times 10^9$$

$$k_{CD} = \frac{EI}{2L} = 1.92 \times 10^9 \times 0.5$$
$$= 0.96 \times 10^9$$

$$k_{AE} = \frac{3EI}{4L} = 0.75 \times 8750 \times 2 \times \frac{1000 \times 102.5^3}{12} \times \frac{1}{2525}$$
$$= 0.46 \times 10^9$$

$$k_{BF} = k_{CG} = \frac{3EI}{4L} = 0.75 \times 8750 \times \frac{1000 \times 215^3}{12} \times \frac{1}{2525}$$
$$= 2.15 \times 10^9$$

Distribution factors

31. The vertical design load at a point immediately above the point of lateral support, ie, the floor slab, is considered axial. Therefore, the wall is considered pinned at this level.

Joint A: factor for span AB $= \dfrac{1.92 \times 10^9}{1.92 \times 10^9 + 0.46 \times 10^9} = 0.81$

Therefore factor for column AE $= 1.0 - 0.81 = 0.19$

Joint B: factor for span BA = factor for span BC

$$= \frac{1.92 \times 10^9}{2 \times 1.92 \times 10^9 + 2.15 \times 10^9} = 0.32$$

Therefore factor for column BF $= 1.0 - 2 \times 0.32 = 0.36$

Joint C: factor for span CB $= \dfrac{1.92 \times 10^9}{1.92 \times 10^9 + 0.96 \times 10^9 + 2.15 \times 10^9} = 0.38$

factor for span CD $= 0.38 \times 0.5 = 0.19$

Therefore factor for column CG $= 1.0 - 0.38 - 0.19 = 0.43$

Moment distribution

(Bending moments kNm/m width)

	A			B			C	
	AE	AB	BA	BF	BC	CB	CG	CD
Distribution factor	0.19	0.81	0.32	0.36	0.32	0.38	0.43	0.19
F.M.		−11.1	+11.1		−10.8	+10.8		−10.8
Distribution	+2.2	+ 8.9	− 0.1	−0.1	− 0.1			
Carry over			+ 4.5					
Distribution			− 1.4	−1.7	− 1.4			
Carry over		− 0.7				− 0.7		
Distribution	+0.1	+ 0.6				+ 0.3	+0.3	+ 0.1
	+2.3	− 2.3	+14.1	−1.8	−12.3	+10.4	+0.3	−10.7

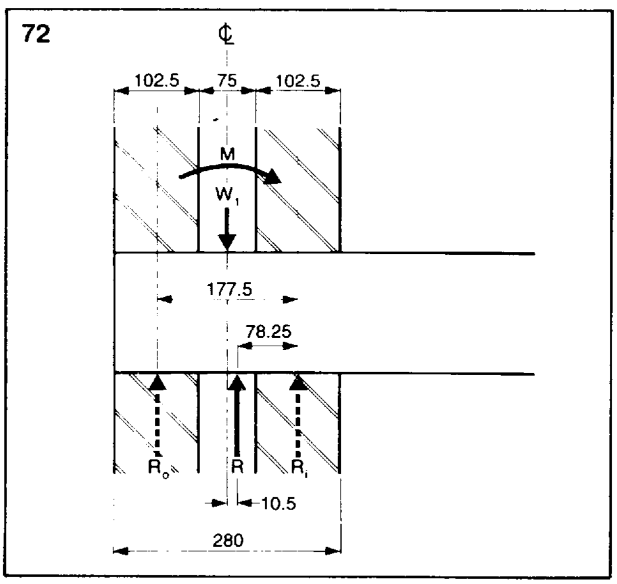

Figure 72 Calculated design eccentricity Wall 1

Design vertical loads at underside of first floor (see Figure 72)

W_1:

Characteristic dead load from inner leaf and floors $=121.4$ kN/m (Table 12)
Self-weight of outer leaf$=2.25 \times 2.6 \times 6$ $= 35.1$ kN/m

156.5 kN/m

Design load, $W_1, = 1.4 \times 156.5 = 219.1$ kN/m
Design bending moment $=2.3$ kNm/m from moment distribution above, at A

Therefore design eccentricity $= \dfrac{2.3 \times 10^6}{219.1 \times 10^3}$

$= 10.5$ mm, see Figure 72

Statically equivalent load on outer leaf, taking moments about R_i (Figure 72):

$R_o = 219.1 \times \dfrac{78.25}{177.5} = 97$ kN/m

and

$R_i = 219.1 - 97 = 122.1$ kN/m

Adding the self-weight of brickwork between ground and first floors:
Total design vertical load on outer leaf
$= 97 + 2.25 \times 2.6 \times 1.4 = 105.2$ kN/m
Total design vertical load on inner leaf
$= 122.1 + 2.5 \times 2.45 \times 1.4 = 130.7$ kN/m

Design wind loads – ground floor level
As both leaves of wall 1 are now loadbearing, the moment of inertia of the wall is increased, and it will attract a higher proportion of the wind moment than originally. However, this is more than offset by the increase in wall area, resulting in slightly lower wind stresses than before. Calculations similar to those on page 77 show that the maximum stress due to characteristic wind loading$=0.27$ N/mm^2. Hence, stress due to design wind loading$=1.4 \times 0.27 = 0.38$ N/mm^2.

Design of inner leaf (now axially loaded)

Design vertical load $= 130.7 + 0.38 \times 102.5$
$= 169.9$ kN/m
Slenderness ratio $= 13.4$, from page 79
Hence for axial load, β $= 0.90$

Design vertical resistance $= \dfrac{\beta\, t\, f_k{}^*}{\gamma_m}$

$= \dfrac{0.9 \times 102.5 \times f_k}{3.5}$

Therefore, equating design load and design resistance and solving:

$f_k = \dfrac{3.5 \times 169.9 \times 10^3}{0.9 \times 102.5 \times 1000} = 6.4$ N/mm^2

From code Figure 1, class 3 bricks in mortar designation (ii), or class 4 bricks in mortar designation (iii), may be used.

* *The narrow wall factor, 1.15, should not be applied to the loaded inner leaf of a cavity wall where both leaves are loaded.*

Design of the outer leaf follows the same procedure.

Wall 2

Characteristic dead load G_k $= 202.1$ kN/m from Table 12
Characteristic imposed load Q_k $= 27.9$ kN/m from Table 12
Design bending stress due to horizontal loads,
$= 0.47$ N/mm^2, see page 78.

22(a) Load case (a) dead and imposed load
$\gamma_f = 1.4$ (dead load) and 1.6 (imposed load)
Design vertical load $= 1.4 \times 202.1 + 1.6 \times 27.9 = 328$ kN/m

22(b) Load case (b) dead and wind load
$\gamma_f = 1.4$ (dead load) and 0.015 (dead load*)

Design vertical load $= 1.4 \times 202.1 + 0.47 \times 102.5 \times \dfrac{10^3}{10^3} = 331$ kN/m

22(c) Load case (c) dead, imposed and wind load
$\gamma_f = 1.2$ (dead load and imposed load) and 0.015 (dead load*)

Design vertical load $= 1.2 \times 202.1 + 1.2 \times 27.9 + 0.47 \times 102.5 \times \dfrac{10^3}{10^3} = 324$ kN/m

* *0,015 G_k exceeds 1.4 W_k and also therefore 1.2 W_k and is used as the design horizontal load (see page 19).*

28. *Slenderness considerations*

28.2.2 Enhanced lateral resistance is provided by wall 8 bonded to wall 2, and by slab bearing on the inner leaf of the wall.

28.3.2 Effective length, h_{ef}, $= 2 \times 493 = 986$ mm
ie $2 \times$ distance between a free edge and a support (wall 8)

28.3.1 Effective height, h_{ef}, $= 0.75 \times 2450 = 1840$ mm

Therefore use the lesser of these values ie 986 mm.

28.4 Effective thickness, t_{ef}, $= 137$ mm, as wall 1.

28.1 Slenderness ratio $= \frac{986}{137} = 7.2$

31. ***Eccentricity at right angles to the wall***

Assume $e_x = 0.02t$ as wall 1 since calculation would be similar to that on page 79, even if giving a slightly different answer.
Hence, from code Table 7, capacity reduction factor, β, $= 1.0$

23.1.1 Horizontal loaded cross-sectional area of wall, A*

$= \frac{1200 \times 102.5}{10^6} = 0.123$ m², ie, less than 0.2 m².

Therefore the characteristic compressive strength of the masonry, f_k, must be multiplied by the factor:
$(0.7 + 1.5A) = (0.7 + 1.5 \times 0.123) = 0.88$.

23.1.2 As the thickness of the inner leaf of wall 2 is equal to the width of a standard format brick, the value of f_k may also be multiplied by 1.15.

32.2.1 ***Design vertical resistance of wall 2***

$$= \frac{\beta t\,(0.88 \times 1.15 f_k)}{\gamma_m} = \frac{1.0 \times 102.5 \times 0.88 \times 1.15 f_k}{3.5}$$

$= 29.6\,f_k$ kN/m, assuming $\gamma_m = 3.5$, as for wall 1.

Design vertical load $= 331$ kN/m (load case (b)).

Therefore, minimum characteristic compressive strength, f_k, required

$= \frac{331 \times 10^3}{29.6 \times 10^3} = 11.2$ N/mm².

From code Figure 1, class 7 bricks in mortar designation (ii), or class 10 bricks in mortar designation (iii) may be used.

33. As wall 2 is not being used to resist wind forces in its plane, there is no need to consider its shear strength.

**It may be argued that it is not necessary to apply the area reduction factor to this wall, as there is an internal wall fully bonded into it; however, strict interpretation of the code does require its application.*

It is usual to rationalise the strengths of brickwork required in a structure, in order to simplify construction and for reasons of economy. In this building, wall 2 is the most heavily stressed, and so the strength of brick required for this wall might be used throughout the ground floor.

The strength of brick required will be less in the upper storeys, and calculations would be done at each level until the class of brick required was less than that of the lowest strength brick economically available. For example, consider the characteristic strength of brickwork required for wall 2 just above third floor level:
Characteristic dead load, G_k, $= 106.6 + 6.1 = 112.7$ kN/m, from Table 12
Characteristic imposed load, Q_k, $= 18.1$ kN/m, from Table 12

20. ***Stability***

Design moment at third floor in direction B is based on $0.015\,G_k$.

G_k at third floor $= \frac{4}{7}(3870 + 6700) + 1410 + \frac{3}{6} \times 11700$

$= 13300$ kN (loads from page 75).

Therefore $0.015\,G_k = 200$ kN, and the design moment $= 200 \times \frac{10.4}{2}$

$= 1040$ kNm.

Thus, the design moment per complex (Figure 69) $= \frac{1040}{2}$

$= 520$ kNm.

The maximum design bending stress in wall 2

$$=\pm\frac{My}{I}=\frac{520\times4.415\times10^6}{15317\times10^6}$$

$=\pm0.150$ N/mm²

22. Load case (a) dead and imposed load:
Design vertical load $=1.4\times112.7+1.6\times18.1$
$=187$ kN/m.
Load case (b) dead and wind load:

Design vertical load $=1.4\times112.7+0.150\times102.5\times\frac{10^3}{10^3}$

$=173$ kN/m.
Load case (c) dead, imposed and wind load:

Design vertical load $=1.2\times112.7+1.2\times18.1+0.150\times102.5\times\frac{10^3}{10^3}$

$=172$ kN/m.
Thus case (a) is the critical case.

28. *Slenderness considerations*

28.1 Slenderness ratio $=7.2$, as for ground floor.

31. *Eccentricity at right angles to the wall*
For simplicity, assume eccentricity, e_x, $=0.02$ t, as for the ground floor.
Hence $\beta=1.0$ from code Table 7.
Area reduction factor, as before, $=0.88$.

32.2.1 *Design vertical resistance of wall 2*

$$=\frac{\beta\, t\, (0.88\times1.15\, f_k)}{\gamma_m}=29.6\, f_k \text{ kN/m}$$

Therefore, the minimum characteristic strength, f_k, required

$$=\frac{187\times10^3}{29.6\times10^3}=6.3 \text{ N/mm}^2$$

From code Figure 1, a class 3 brick in mortar designation (ii) may be used, or a class 4 brick in mortar designation (iii). The same brick may also be used for the other brickwork at this level, and above if required.

Wall 3

Characteristic dead load, G_k, $=289.9$ kN/m, from Table 12
Characteristic imposed load, Q_k, $=35.7$ kN/m, from Table 12
Stress due to characteristic wind load $=\pm0.15$ N/mm² or ±0.12 N/mm², from Table 14.

22(a) Load case (a) dead and imposed load:
$\gamma_f=1.4$ (dead load) and 1.6 (imposed load).
Design vertical load $=1.4\times289.9+1.6\times35.7$
$=463$ kN/m.

22(b) Load case (b) dead and wind load:
$\gamma_f=1.4$ (dead load) and 1.4 (wind load).

Design vertical load $=1.4\times289.9+1.4\times0.15\times215\times\frac{10^3}{10^3}$

$=450$ kN/m.

22(c) Load case (c) dead, imposed and wind load:
$\gamma_f=1.2$ (dead, imposed and wind load).

Design vertical load $=1.2\times289.9+1.2\times35.7+1.2\times0.15\times215\times\frac{10^3}{10^3}$

$=430$ kN/m.

28. *Slenderness considerations*
28.2.2 Enhanced lateral resistance as wall 1 (see page 79).
28.3.1 Effective height, h_{ef}, $=0.75\times2450=1840$ mm.
28.4 Effective thickness, t_{ef} $=215$ mm.

28.1 Slenderness ratio $=\frac{1840}{215}$ $=8.55$.

31. *Eccentricity at right angles to the wall*
$e_x=0$, as wall is symmetrically loaded, but see page 57 for discussion of the effect of varying loading on each side of internal wall.
Hence, from code Table 7, capacity reduction factor, β, $=0.99$.

32.2.1 *Design vertical resistance of wall 3*

$$=\frac{\beta\, t\, f_k}{\gamma_m}=\frac{0.99\times215\times f_k}{3.5}$$

$=60.8\ f_k$ kN/m, assuming $\gamma_m=3.5$ as for wall 1.
Design vertical load $=463$ kN/m (load case (a)).
Therefore minimum characteristic compressive strength, f_k, required

$$=\frac{463\times10^3}{60.8\times10^3}=7.6\ \text{N/mm}^2.$$

From code Figure 1, class 4 bricks in mortar designation (i), or class 5 bricks in mortar designation (iii) may be used.

33. *Horizontal shear force in plane of wall*

Characteristic shear load due to wind on wall
$=3.6$ kN, from Table 13.
Characteristic dead load
$=289.9$ kN, from Table 12.
Loading case (b) gives the most severe conditions
Design wind load $=1.4\times W_k$
Design dead load $=0.9\times G_k$

Therefore, design shear stress, v_h, $=\frac{1.4\times3.6\times10^3}{2227\times215}$
$=0.010$ N/mm².

25. Characteristic shear strength of masonry, f_v,
$=0.35+0.6\ g_A$ for walls in mortar designation (i), (ii) or (iii).
Design vertical load per unit area, g_A, $=\frac{0.9\times289.9\times10^3}{215\times1000}$
$=1.22$ N/mm².
Therefore, f_v, $=0.35+0.6\times1.22$
$=1.08$ N/mm².

27.4 The partial safety factor for shear loads, γ_{mv}, $=2.5$.
Therefore, the design shear resistance of the wall

$$=\frac{f_v}{\gamma_{mv}}=\frac{1.08}{2.5}=0.43\ \text{N/mm}^2.$$

This exceeds the design shear stress, v_h, and is therefore satisfactory.

Parallel calculation

31. In the design of wall 3, the eccentricity at right angles to the wall is zero, as the loading is symmetrical from each side. Had the spans on each side of the wall differed greatly and were the Q_k on one side to be taken as zero, a further load case might have had to be considered. However, according to the code, the imposed load, Q_k, is only taken as zero in the dead+wind load combination. The calculation below shows what would happen if Q_k was taken as zero on the short span side, following the method for wall 3, see Figure 73.

73

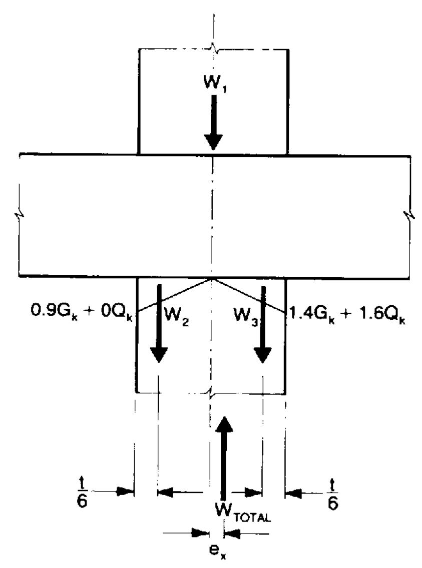

Figure 73 Loading case with zero live load on one span.

Loading (see page 73)

$W_2 = 0.9 \times 30.3 \times 0.5 \quad = 13.6$ kN/m

$W_3 = 1.4 \times 30.3 \times 0.5 + 1.6 \times 8.8 \times 0.5 = 28.2$ kN/m

Therefore, $W_1 = 463 - 2 \times 28.2 \quad = 406.6$ kN/m

448.4 kN/m

Taking moments about the centre line:

$$448.4\, e_x = (28.2 - 13.6)\, \frac{215}{3}$$

Therefore, $e_x = 2.33$ mm $= 0.01$ t

Thus, from code Table 7, the load may be considered axial and the critical design case will still be full load on both spans.

23.1.2 If, for example, wall 3 had been made equal in thickness to the width of the bricks, ie, a narrow brick wall, and the bricks were of standard format, then its design would be modified as follows:

28. ***Slenderness considerations***

28.3.1 Effective height, h_{ef}, $= 0.75 \times 2450$

$= 1840$, as before.

28.4 Effective thickness, t_{ef}, $= 102.5$ mm.

28.1 Slenderness ratio $= \dfrac{1840}{102.5}$

$= 18.0$

31. ***Eccentricity at right angles to the wall***

$e_x = 0$. From code Table 7, capacity reduction factor, $\beta, = 0.77$

32.2.1 ***Design vertical resistance of wall***

23.1.2 As the wall is narrow, the characteristic compressive strength, f_k, may be multiplied by 1.15. Thus, design vertical resistance

$$= \frac{\beta\, t \times 1.15\, f_k}{\gamma_m} = \frac{0.77 \times 102.5 \times 1.15\, f_k}{3.5}$$

$= 25.9\, f_k$.

Design vertical load $= 1.4\,(289.9 - 2.5 \times 17.2) + 1.6 \times 35.7$

$= 402.8$ kN/m.

Therefore, minimum f_k required $= \dfrac{402.8 \times 10^3}{25.9 \times 10^3}$

$= 15.6$ N/mm^2

From code Figure 1, a class 8 brick in mortar designation (i), or a class 11 brick in mortar designation (ii), could be used. These are higher classes than might normally be used, but if the special category of manufacturing and construction control can be achieved (see page 61), f_k becomes $15.6 \times \dfrac{2.5}{3.5} = 11.1$ N/mm^2 and, from code Figure 1. a class 5 brick in mortar designation (i) can be used.

Wall 4

Characteristic dead load, G_k, $= 357.8$ kN, from Table 12.

Characteristic imposed load, Q_k, $= 50.7$ kN, from Table 12.

As wall 4 is not required to resist horizontal forces, only load case (a) need be considered.

22(a) Load case (a) dead and imposed load:

$\gamma_f = 1.4$ (dead load) and 1.6 (imposed load)

Design vertical load $= 1.4 \times 357.8 + 1.6 \times 50.7$

$= 582$ kN.

3.7 The wall length (800 mm) is less than 4 times its thickness (215 mm), and hence the wall must be considered as a column for design purposes.

28. ***Slenderness considerations***

Lateral support is provided by the concrete slab in both directions.

28.3.1.2 Effective height, h_{ef}, $= 2600$ mm.

28.4.1 Effective thickness, t_{ef}, $= 215$ mm.

28.1 Slenderness ratio $= \dfrac{2600}{215} = 12.1$

31. ***Eccentricity at right angles to the wall***

$e_x = 0$, as the wall is symmetrically loaded, but see page 57 for discussion of the effect of varying loading from each side of internal walls.
Hence from code Table 7, capacity reduction factor, β, $= 0.93$.

23.1.1 Horizontal loaded cross-sectional area of wall, A,

$$= \frac{800 \times 215}{10^6}$$
$$= 0.172 \text{ m}^2$$

Therefore, the characteristic compressive strength of the masonry must be multiplied by the factor: $(0.7 + 1.5A) = (0.7 + 1.5 \times 0.172)$
$= 0.96$

32.2.1 ***Design vertical resistance of wall 4***

$$= \frac{\beta \, b \, t \times 0.96 \, f_k}{\gamma_m} = \frac{0.93 \times 0.8 \times 215 \times 0.96 \, f_k}{3.5}$$

$= 43.9 \, f_k$ kN, assuming $\gamma_m = 3.5$, as wall 1.
Design vertical load
$= 582$ kN
Therefore minimum characteristic compressive strength, f_k, required

$$= \frac{582 \times 10^3}{43.9 \times 10^3}$$
$$= 13.3 \text{ N/mm}^2$$

From code Figure 1, class 7 bricks in mortar designation (i) may be used.

37. **Design: Accidental damage**

The example building falls into Category 2, having five storeys or more, thus the additional detailed recommendations in code clause 37 for the limitation of accidental damage must be met over and above the recommendations in code clause 20.2 for the preservation of structural integrity.

Three options are listed in code Table 12 for these additional recommendations, one of which must be adopted.

Option 1 is the full engineering option and requires the designer to prove that all vertical and horizontal elements are removable one at a time, without causing collapse unless the member being considered is protected. A protected member is one that is able to withstand, together with its essential supports, its reduced design loads (see clause 22(d)), and an accidental load of 34 kN/m², applied in any direction.

Option 3 is the 'soft' option which lays down rules for horizontal ties and vertical ties. Compliance with these rules removes the need for further consideration of residual stability or spread of damage.

Option 2 combines the horizontal tie requirements of Option 3 and the need to prove the vertical elements removable, unless protected as in Option 1. Option 2 will be used, in this example without consideration of the horizontal elements, as these are not the subject of this handbook.

Table 12 ***Option 2***

Horizontal ties

Table 13 Basic horizontal tie force, $F_t = 60$ kN or $20 + 4N_s$ whichever is less where N_s is the number of storeys (including ground and basement):
$20 + 4N_s = 20 + 4 \times 7 = 48$ kN
This is less than 60kN, therefore use $F_t = 48$ kN

Peripheral, internal and external wall ties are required at each floor level and roof level, as shown in Figure 74. They may be provided by using the reinforcement required for the floor and roof slabs. The internal ties may be spread over the full width of the building or concentrated at maximum 6.0m centres. The peripheral ties must be placed within 1.2m of the edge of the floor or roof. Internal ties must all be fully anchored to the perimeter ties. The tie connection to masonry external walls may be based on shear strength or friction at the wall/slab interface, but not on both.

Table 13 Design tie forces: (using formulae given in code Table 13)
Peripheral ties: design tie force $= F_t = 48$ kN.
Internal ties: design tie force $= \dfrac{F_t \, (G_k + Q_k)}{7.5} \times \dfrac{L_a}{5}$ or F_t, whichever is the greater.

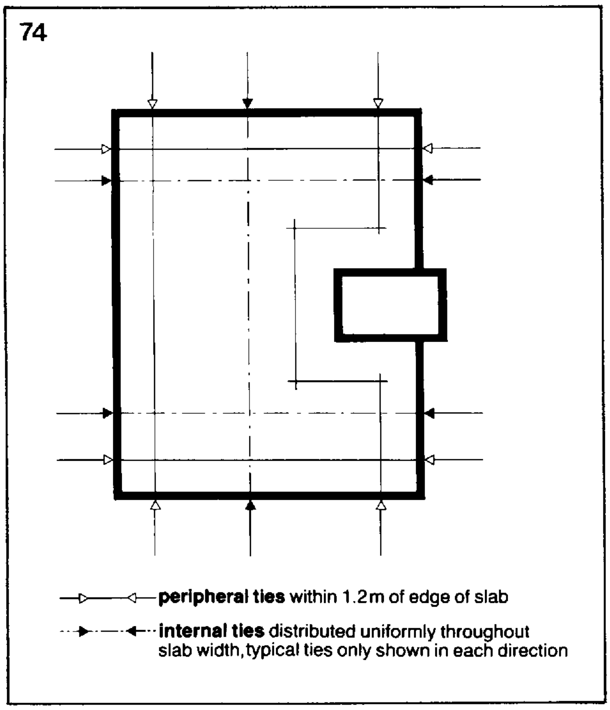

Figure 74 Typical floor plan showing peripheral and internal ties to comply with option 2 and 3 requirements for horizontal tying.

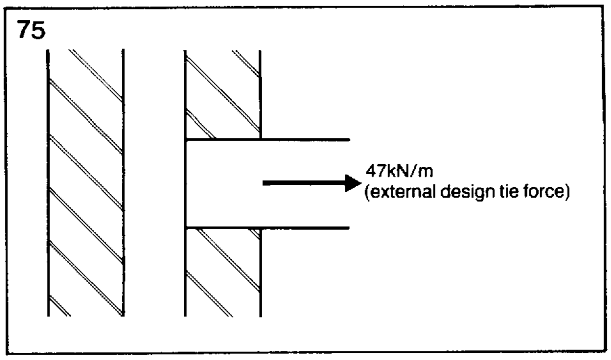

Figure 75 Tying to external masonry walls.

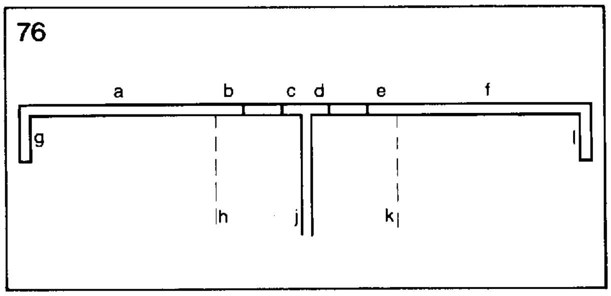

Figure 76 Walls on line A

$$\frac{F_t\,(G_k+Q_k)}{7.5} \times \frac{L_a}{5} = \frac{48\,(6.9+2.0)}{7.5} \times \frac{3.67}{5}$$

$=42$ kN.

Therefore, use $F_t=48$ kN.

Ties to external walls: design tie force $= \left(\frac{h}{2.5}\right) F_t$ or $2\,F_t$, whichever is the lesser.

$$\left(\frac{h}{2.5}\right)F_t = \frac{2.45}{2.5} \times 48$$

$=47$ kN/m.

This is less than $2F_t$, therefore design tie force$=47$ kN/m, see Figure 75.

Tying connections to external walls are required along all external loadbearing walls.

Check shear strength of masonry to concrete connection.

Design shear stress at masonry/concrete interface $= \frac{47 \times 10^3}{2 \times 102 \times 1000}$

$=0.23$ N/mm^2.

27.4 Minimum design shear strength

$$= \frac{0.35}{\gamma_{mv}}$$

γ_{mv} may be reduced to 1.25 when considering the effects of accidents.

Therefore, minimum design shear strength$=\frac{0.35}{1.25}$

$=0.28$ N/mm^2.

This exceeds 0.23 N/mm^2, and external wall tie requirements are therefore satisfied.

Table 12 *Vertical ties* (*these requirements would also apply to option 1, had it been chosen*)
Vertical elements must be proved removable, one at a time, without causing collapse, unless they are protected (see page 68). In the lower storeys, the vertical members can be shown to be protected; the upper storey members must be shown to be removable.

Walls on line A (from figure 60).

37.5 Partitions h and k in Figure 76 are substantial partitions having a weight of not less than 150 kg/m^2, and walls g and 1 have a length without openings of not less than the clear height of the wall between horizontal lateral supports divided by 2, = 1250 mm, thus they may be assumed to provide lateral support to the wall element provided that the edge strip of the slab above is designed to span from left to right onto walls g to h to j etc., any one of walls a to f can be removed.

The edge strip of slab at each floor level must be designed to carry one storey height of brickwork, plus the floor loading with the reduced partial safety factors permitted in CP 110. The load on the edge strip of the slab can be reduced by designing the whole slab panel to span between g and j, and j and l.

37.1.1 If walls g and l had been less than 1250 mm long a small reinforced concrete column would need to be introduced at the junction of walls g and a and also f and l. The column would then be designed as a protected member, ie to carry 34 kN/m² in any direction on both itself and the walls it supports.

Walls on line B (from Figure 60)

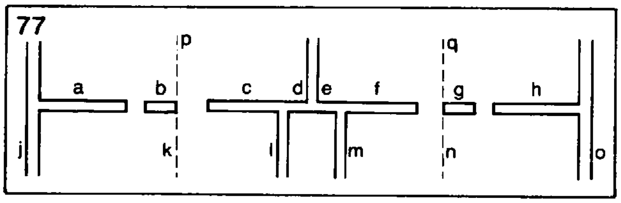

Figure 77 Walls on line B

Walls j and o in Figure 77 have a length without openings greater than 1250 mm and walls k, n, p and q are substantial partitions as before; again, the removal of any one of walls a to h is treated in a similar manner to the walls on line A.

Walls on line C (from Figure 60)

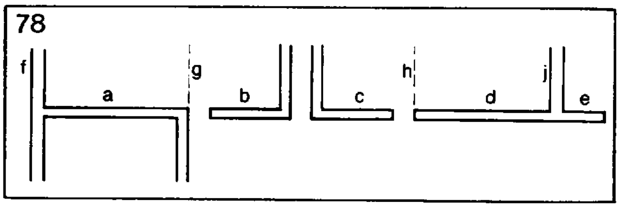

Figure 78 Walls on line C

Walls f and j in Figure 78 have a length, without openings, greater than 1250 mm, and walls g and h are substantial partitions, as before, so again the removal of any one of the walls a to e is treated in a similar manner to the walls on line A.

37.1.1 ***Protected walls***

For walls to be considered as protected, they must be capable of resisting an accidental design load of 34 kN/m².
Consider wall 3 at underside of 4th floor.

Design loads (Characteristic loads taken from Table 12)

22. Load case (d) code clause 22,

design dead load $= \gamma_f G_k = 0.95 \times 149.8 \qquad = 142.3$ kN/m

design imposed load

(no imposed load reduction) $= \gamma_f Q_k = 0.35\,(3 \times 8.8 + 6.6) = 11.6$ kN/m

$= 153.9$ kN/m

36.8 The design lateral strength of the wall, q_{lat},

37.1.1 $= \dfrac{8\,t\,n}{h^2 \gamma_m}$, where $\gamma_m = 1.05$ (code clause 37.1.1)

$$= \frac{8 \times 0.215 \times 153.9}{2.45^2 \times 1.05}$$

$= 42$ kN/m² (> 34 kN/m²)

$$\frac{\text{height}}{\text{thickness}} = \frac{2450}{215}$$

$= 11.4$ (< 20).

Hence, wall 3 is capable of resisting the accidental design load of 34 kN/m² below 4th floor level, and can therefore be considered as a protected member below this level. It is thus only necessary to consider its removal above the 4th floor.

A similar check can be made for the other vertical elements.

Design of external walls subjected to lateral loading

Assuming that wind forces on the windows of the building are not transferred to the adjacent brickwork, ie that frames are full storey height and span top to bottom, only wall 1 (Figure 60) at top floor level need be designed to resist lateral wind loads. Below the top storey there will be, by inspection, sufficient precompression on the walls to resist wind forces. Also by inspection other walls at top floor level are small enough to resist wind forces.

The wall is simply supported top and bottom and full continuity exists at the sides, ie around the corner of the building and over the internal partition which is block bonded into the inner leaf, see Figure 79.

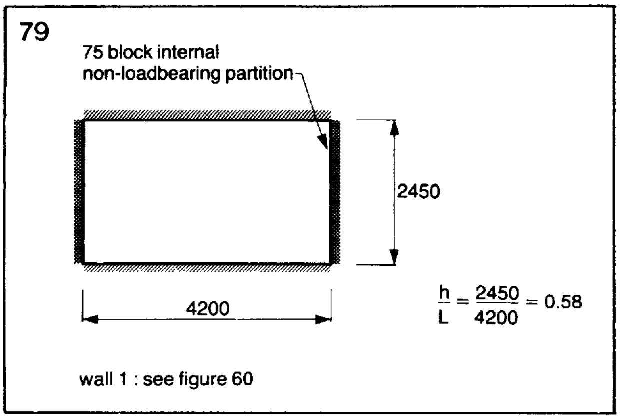

Figure 79 Laterally loaded wall.

36.3 *Limiting dimensions*

For a panel supported on four sides with less than three sides continuous, the height × length of the panel must be less than or equal to $2025 \times t_{ef}^2$:

$t_{ef} = 0.67\ (2 \times 102.5) \quad = 137.3$ mm

Therefore $2025 \times 137.3^2 = 38.17 \times 10^6$ mm²

$h \times L = 2450 \times 4200 \quad = 10.29 \times 10^6$ mm²

This is less than $2025 \times t_{ef}^2$ and therefore satisfactory.

No wall dimension may exceed $50 \times t_{ef}$

$= 50 \times 137.3 = 6865$ mm.

Wall length = 4200 mm, therefore the wall satisfies the limiting dimension clause.

Wind loading

Dynamic wind pressure, q, =0.875 kN/m² (see page 72).

Pressure coefficients

Building dimension ratios:

$$\frac{\text{height}}{\text{width}} = \frac{18.2}{15.175} = 1.2, \frac{\text{length}}{\text{width}} = \frac{18.67}{15.175} = 1.23.$$

Width of building affected by local pressure $= 0.25 \times 15.175 = 3.8$ m.

Therefore, from CP3: Chapter V: part 2 Table 7,

local $C_{pe} = -1.1$

$C_{pi} = -0.3$ or $+0.2$

Therefore maximum C_{pe}-$C_{pi} = -1.3$

Therefore characteristic wind load, W_k, $= 1.3 \times 0.875$

$= 1.14$ kN/m²

Design wind load, $\gamma_f W_k = 1.4 \times 1.14 \quad = 1.6$ kN/m²

γ_f is taken as 1.4 as the wall contributes to the overall stability of the building.

24. ***Characteristic flexural strength***

Both leaves of the wall are assumed to be of clay bricks, having a water absorption of over 12%, in mortar designation (i). Thus, from code Table 3, the characteristic flexural strengths, f_{kx}, of the masonry are 0.4 N/mm² and 1.1 N/mm² in the two orthogonal directions.

36.4.2 The roof load is carried on the inner leaf, and this load together with the self-weight of the top half of the inner leaf acts to increase its flexural strength in the parallel* direction. Similarly, the self-weight of the top half of the outer leaf acts to increase its flexural strength in the parallel direction.

36.4.5 As both leaves have the same moment of resistance in the perpendicular** direction, being of the same materials, the applied horizontal force must be shared equally between them. For this reason, it is necessary only to check the strength of the outer leaf, since if it is satisfactory, the inner leaf, with the greater vertical load, will automatically be so.

Design vertical dead load due to self-weight of top half of outer leaf

$= 0.9 \times 1.2 \times 2.25 \quad = 2.4$ kN/m run, note γ_f is taken as 0.9.

Stress due to design vertical load

$$= \frac{2.4 \times 10^3}{102.5 \times 1000} = 0.023 \text{ N/mm}^2$$

* *Bending in the 'parallel' direction refers to bending about an axis parallel to the plane of the bed joint.*

***Bending in the 'perpendicular' direction refers to bending about an axis perpendicular to the plane of the bed joint.*

Therefore the modified orthogonal ratio, μ,
$=\frac{0.4+0.023\times 3.5}{1.1}=\frac{0.48}{1.1}=0.44.$

36.4.2 Design moment in panel:
The bending moment coefficient, α, for the panel shape shown in Figure 79, for an orthogonal ratio of 0.44, from code Table 9G is 0.025. Thus, the design bending moment on the outer leaf in the perpendicular direction

$$=\alpha W_k \gamma_f L^2=0.025\times\frac{1.4\times 1.14}{2}\times 4.2^2=0.35 \text{ kNm/m.}$$

36.4.5 The design moment of resistance of each leaf in the perpendicular direction
$=\frac{f_{kx}}{\gamma_m}Z=\frac{1.1}{3.5}\times\frac{1000\times 102.5^2}{6}\times\frac{1}{10^6}=0.55 \text{ kNm/m.}$
This exceeds the design bending moment and is therefore satisfactory.

Parallel calculation

Alternative approach to design for lateral load – effective eccentricity

80

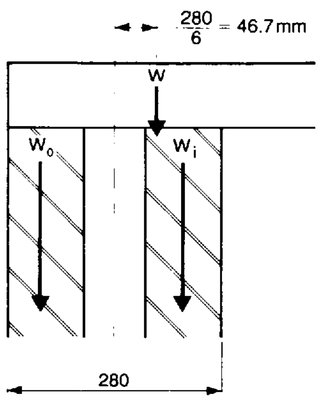

Figure 80 Effective eccentricity.

The code assumes in Appendix B that the total design eccentricity of vertical loading is usually a maximum in the mid-height region of the wall.
Therefore, considering the bending moment due to lateral wind forces at mid-height of the wall to be $\frac{W_k L^2}{8}$, the design bending moment
$=\frac{1.6\times 2.6^2}{8}=1.35 \text{ kNm/m.}$

The code does not give any guidance as to how this moment should be shared by the leaves in this case, but from engineering principles it would be appropriate to share it in proportion to the stiffness of the two leaves. In this instance, the leaves are of the same stiffness and, therefore, each has to resist half the moment.

22(c) Consider wall 1, loading case $1.4G_k$, $1.4W_k$:
The roof design vertical load
$=1.4G_k=1.4\times 9.2$
$=12.9$ kN.

31. This is considered to act at an eccentricity of 46.7 mm (see Figure 80). Taking moments about the centre line of the outer leaf, the statically equivalent axial load, W_i, on the inner leaf
$=\frac{12.9\times 135.45}{177.5}$
$=9.8$ kN.
Therefore, $W_o=12.9-9.8$
$=3.1$ kN.

Consider the outer leaf:
Effective eccentricity due to lateral load $=\frac{\text{design bending moment on wall}}{\text{design axial vertical load}}$

$$=\frac{\frac{1.35}{2}\times 10^6}{3.1\times 10^3}=218 \text{ mm}=2.12 \text{ t.}$$

This is obviously unacceptable, and there is no point in proceeding with this approach. It can be seen that, in order to reduce the effective eccentricity to a sensible figure (0.499 recurring x t being the theoretical maximum), a considerable increase in the vertical load on the outer leaf is required. For this reason, this method is rarely a practical alternative for the design of laterally loaded wall panels with little vertical load. Even when the vertical load is larger, panels are more conveniently designed using the vertical arching approach given in code clause 36.8.

EXAMPLE 2
Three-storey end of terrace house

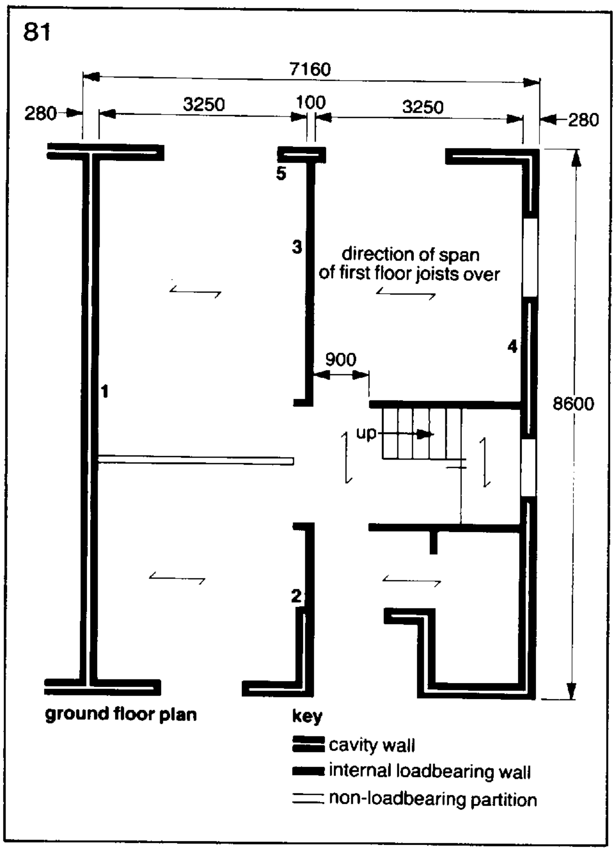

Figure 81 Ground floor plan.

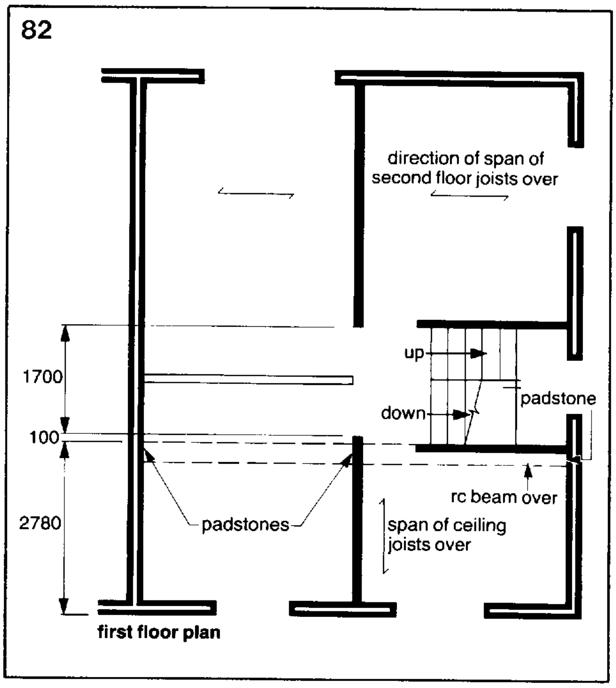

Figure 82 First floor plan.

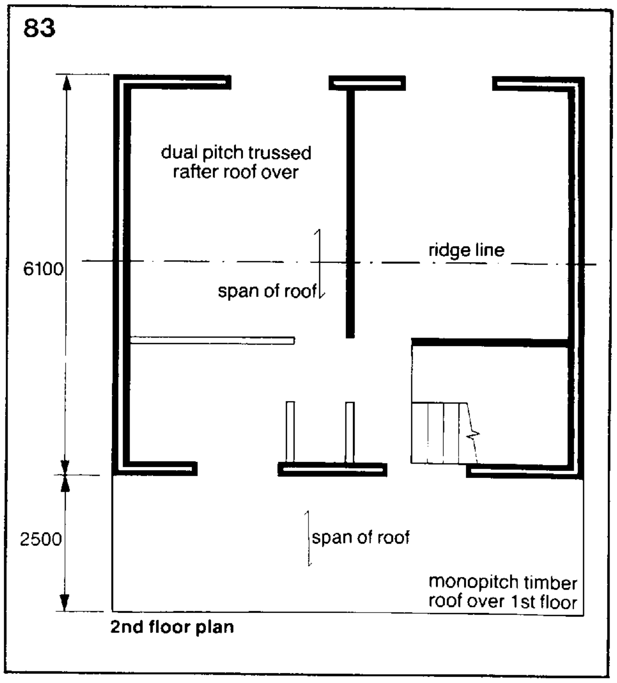

Figure 83 2nd floor plan.

Floor plans and cross sections of an end of terrace three storey house are shown in Figures 81 to 87. The external cavity walls are 280 mm thick with a facing brick outer leaf, a 100 mm lightweight block inner leaf, and cavity insulation. The party wall is a 280 mm cavity brick wall and the internal load bearing partitions are either 100 mm blockwork or 102.5 mm brickwork.

The floor construction is timber boarding on timber joists either supported on joist hangers or built into the masonry. The roof construction is tiles and battens on timber trussed rafters supported on wall plates on the inner leaf of the front and rear walls.

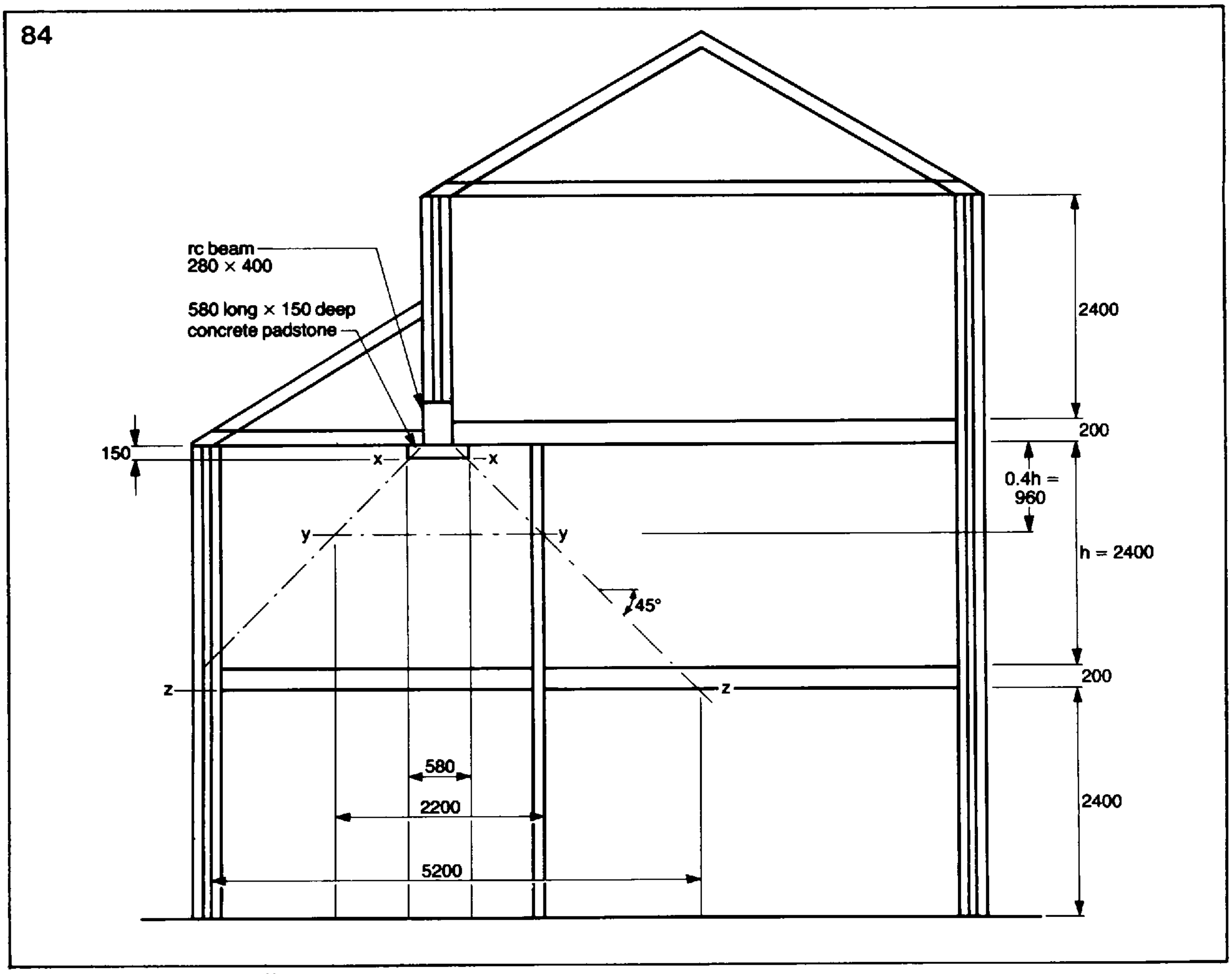

Figure 84 Elevation on wall 1.

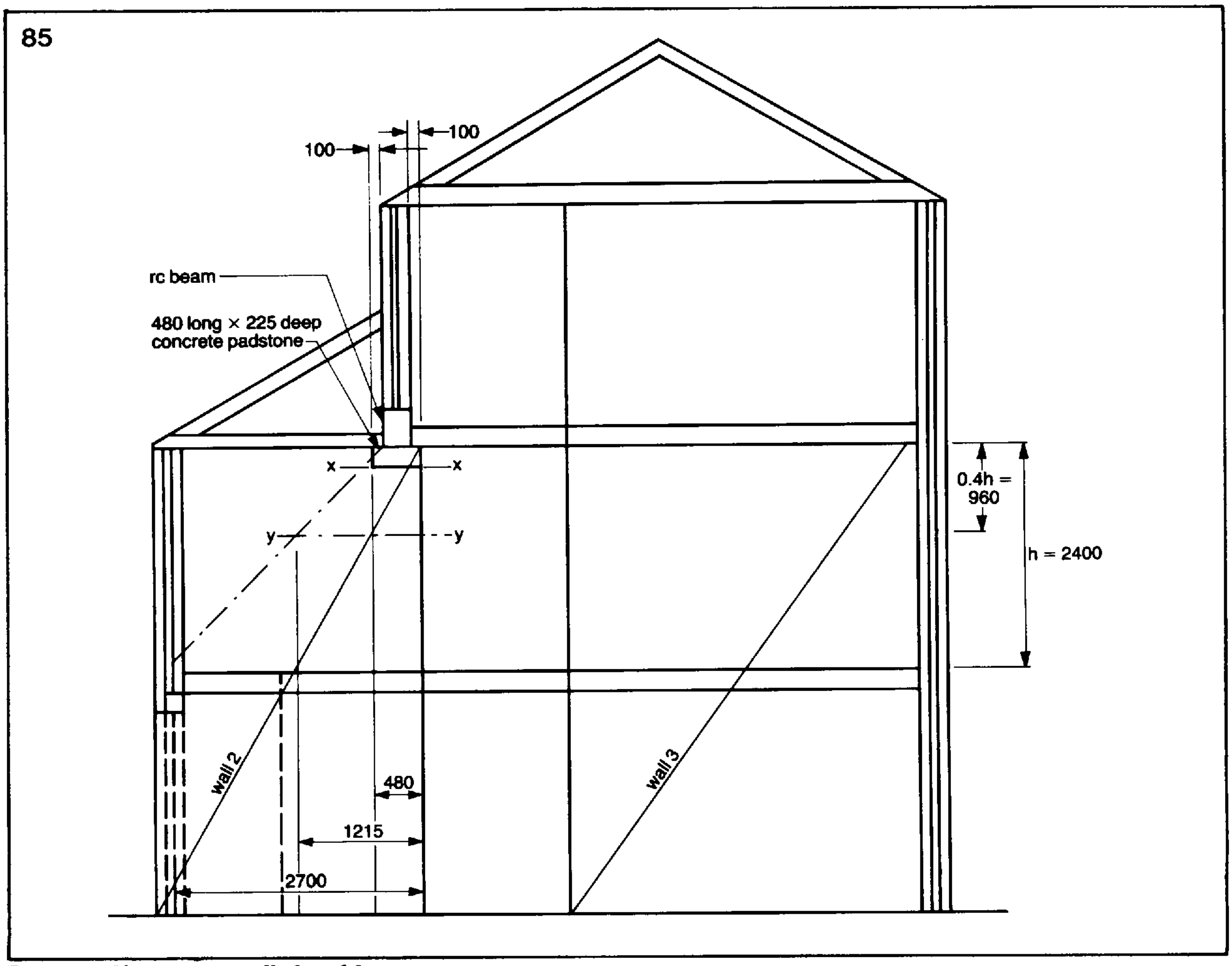

Figure 85 Elevation on walls 2 and 3.

86

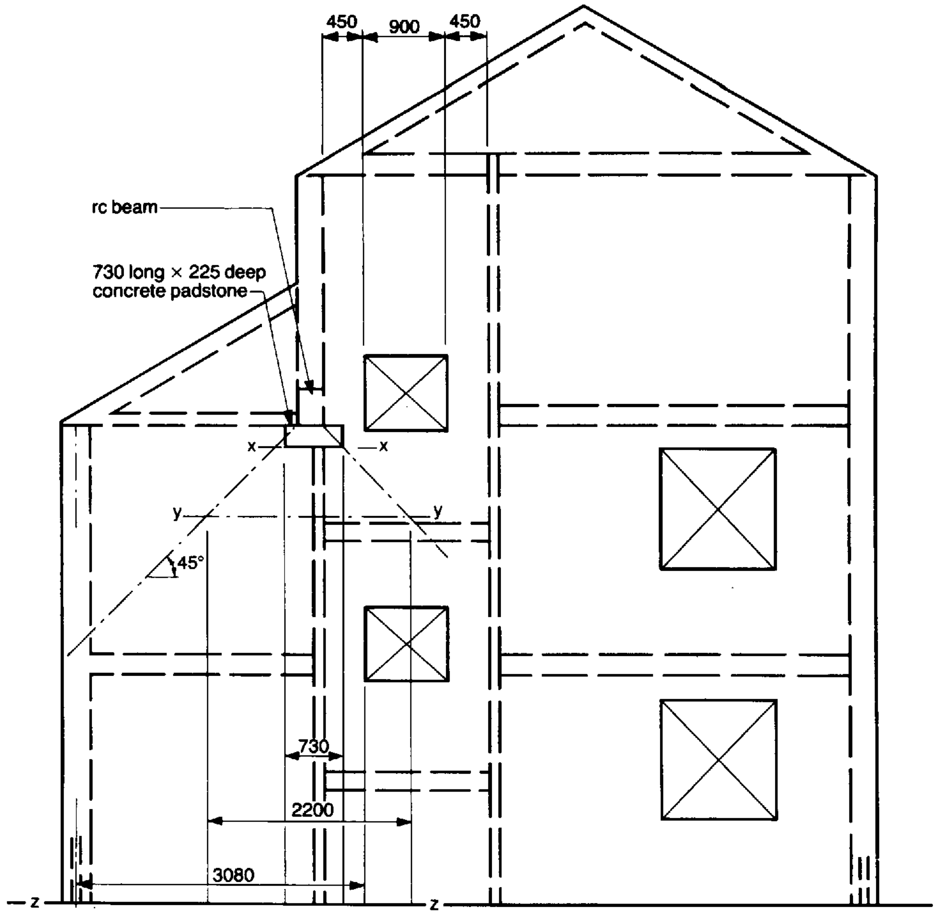

Figure 86 Elevation on wall 4.

87

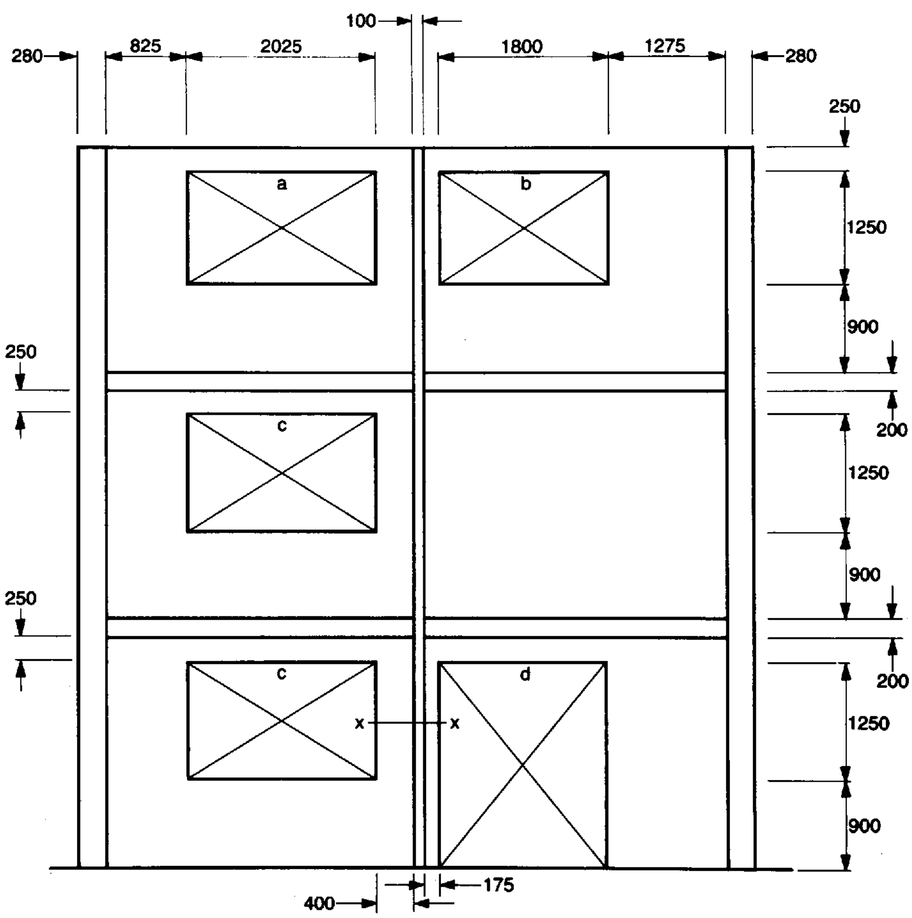

Figure 87 Elevation on wall 5.

It is assumed that the roof is adequately braced to act as a rigid diaphragm to resist horizontal loads, and that the floors and roof are adequately fixed to the masonry with connections capable of providing at least simple resistance to lateral movement, in accordance with Appendix C to BS 5628: Part 1: 1978.

Loading
From CP 3: Chapter V: Part 1: 1972

2.1 ***Characteristic loads***

Roof load (on plan)

Dead loads, G_k:	tiles	0.58 kN/m²
	felt and battens	0.12 kN/m²
	trussed rafters	0.25 kN/m²
	ceiling	0.15 kN/m²
		1.10 kN/m²
Imposed load, Q_k:		0.75 kN/m²

Floor load

Dead loads, G_k:	boarding	0.12 kN/m²
	floor joists	0.13 kN/m²
	ceiling	0.15 kN/m²
	partitions (lightweight)	0.50 kN/m²
		0.90 kN/m²
Imposed load, Q_k:		1.50 kN/m²

Self-weight of walls, G_k

External wall:	102.5 mm brick outer leaf	2.25 kN/m²
	100 mm lightweight block plastered one side	1.15 kN/m²
	280 mm cavity wall plastered one side	3.40 kN/m²
Party wall:	cavity wall with 102.5 mm brick leaves plastered both sides	5.00 kN/m²
Internal partition walls:		
	100 mm lightweight block plastered both sides	1.40 kN/m²
	102.5 mm brick plastered both sides	2.75 kN/m²

Wind loading (from CP 3: *Chapter V*: *Part* 2: 1972)
Assume basic wind speed = 40 m/s,
and wind speed factors:
$S_1 = 1.0$
$S_2 = 0.74$ using ground roughness category (3) class B,
height of building = 9.75 m to ridge
$S_3 = 1.0$
Therefore, design wind speed, V_s, $= 40.0 \times 1.0 \times 0.74 \times 1.0 = 29.6$ m/s
Therefore, dynamic wind pressure, $q, = \dfrac{0.613 \times 29.6^2}{10^3} = 0.537$ kN/m²

Design of walls for vertical load
Walls numbered 1 to 5 on plan in Figure 81 will be designed.

Wall 1, 280 mm cavity party wall
An elevation on the wall is shown in Figure 84. The critical sections for design are x-x, y-y and z-z. Whilst the critical section for design of a uniformly loaded wall is usually 0.4h below the level of lateral restraint, for simplicity, section z-z will be designed here, allowance being made for the additional self-weight of the wall, see page 100 for further explanation.

Section x-x (580 mm long) under padstone
Characteristic loading

		G_k (dead load)	Q_k (imposed load)
Load on rc beam:			
roof G_k	$= 1.1 \times 8.6 \times 0.5$	= 4.7 kN/m	
Q_k	$= 0.75 \times 8.6 \times 0.5$		= 3.2 kN/m
wall self-weight	$= 3.4 \times 2.2$	= 7.5 kN/m	
beam self-weight	$= 24 \times 0.4 \times 0.28$	= 2.7 kN/m	
		14.9 kN/m	3.2 kN/m

Load from 2nd floor:

floor G_k	$= 0.9 \times 3.25 \times 0.5$	$= 1.5$ kN/m	
Q_k	$= 1.5 \times 3.25 \times 0.5$		$= 2.4$ kN/m

Considering loads on one leaf only of the party wall, total load at section x-x:

rc beam G_k	$= 14.9 \times 3.25 \times 0.5$	$=24.2$ kN	
Q_k	$= 3.2 \times 3.25 \times 0.5$		$= 5.2$ kN
2nd floor G_k	$= 1.5 \times 0.15$	$= 0.2$ kN	
Q_k	$= 2.4 \times 0.15$		$= 0.4$ kN
wall self-weight	$= 5.0 \times 0.5 \times 3.0 \times 0.58$	$= 4.4$ kN	
		28.8 kN	5.6 kN

Design loading at x-x

$1.4\,G_k = 1.4 \times 28.8 = 40.3$ kN
$1.6\,Q_k = 1.6 \times 5.6 = 9.0$ kN
49.3 kN

Stress due to design load

$$= \frac{49.3 \times 10^3}{102.5 \times 580} = 0.83 \text{ N/mm}^2.$$

34. The padstone bearing conforms to bearing type 1 (code Figure 4). The narrow wall factor, 1.15, is not applied to f_k here, because code clause 23.1.2 refers specifically to the loaded inner leaf of cavity walls, and should not be used when both leaves are loaded.

Therefore, the design strength $= \dfrac{1.25\,f_k}{\gamma_m}$

Assuming normal manufacturing and construction control, from code Table 4 $\gamma_m = 3.5$

Hence, $\dfrac{1.25\,f_k}{3.5}$ must be greater than 0.83 N/mm².

Therefore, minimum f_k required $= \dfrac{0.83 \times 3.5}{1.25} = 2.32$ N/mm².

Section y-y (2200 mm long) 0.4h below underside of rc beam

Characteristic loading

		G_k (dead load)	Q_k (imposed load)
rc beam G_k	$= 24.3 \times \dfrac{1.0}{2.2}$	$=11.0$ kN/m	
Q_k	$= 5.3 \times \dfrac{1.0}{2.2}$		$= 2.4$ kN/m
2nd floor G_k	= as section x-x	$= 1.5$ kN/m	
Q_k	= as section x-x		$= 2.4$ kN/m
wall self-weight	$= 5.0 \times 0.5 \times 3.96$	$= 9.9$ kN/m	
		22.4 kN/m	4.8 kN/m

Design loading at y-y

$1.4\,G_k = 1.4 \times 22.4 = 31.4$ kN/m
$1.6\,Q_k = 1.6 \times 4.8 = 7.7$ kN/m
39.1 kN/m

28. ***Slenderness considerations***

28.2.2.1 Simple resistance to lateral movement is provided by timber joist hangers in accordance with code Figure 13 to the right of the rc beam, as viewed in Figure 84; and by the straps in accordance with code Figure 18 to the left of the rc beam.

28.3.1 Effective height, h_{ef}, $= 2400$ mm

28.4 Effective thickness, t_{ef}, $= 0.67\,(2 \times 102.5)$
$= 137$ mm

28.1 Therefore slenderness ratio $= \dfrac{2400}{137}$
$= 17.5$

31. ***Eccentricity at right angles to the wall, see Figure 88***

The reinforced concrete beam is of sufficient stiffness and short enough span to enable its reaction on the wall to be considered as an axial load. The second floor loading is applied 25 mm from the face of the wall, ie,
$102.5 \times 0.5 + 25 = 76.3$ mm eccentricity.

Two loading cases will be checked, as follows:
(a) minimum axial load, $0.9G_{k2}+1.6Q_{k2}$+maximum floor load, $1.4G_{k1}+1.6Q_{k1}$
(b) maximum axial load, $1.4G_{k2}+1.6Q_{k2}$+maximum floor load, $1.4G_{k1}+1.6Q_{k1}$

(a) loading for maximum eccentricity:
$W_1=1.4G_{k1}+1.6Q_{k1}$
$=1.4\times1.5+1.6\times2.4$
$=5.94$ kN/m.
$W_2=0.9G_{k2}+1.6\,Q_{k2}$
$=0.9\,(22.4-1.5)+1.6\,(4.8-2.4)$
$=22.65$ kN/m.
Taking moments about the centre line:
$5.94\times76.3=(22.65+5.94)\,e_x$
Therefore, $e_x=\dfrac{453.2}{28.59}=15.9$ mm
$=0.15\,t$
The slenderness ratio = 17.5
Therefore, by interpolation, from code Table 7,
capacity reduction factor, β, =0.65

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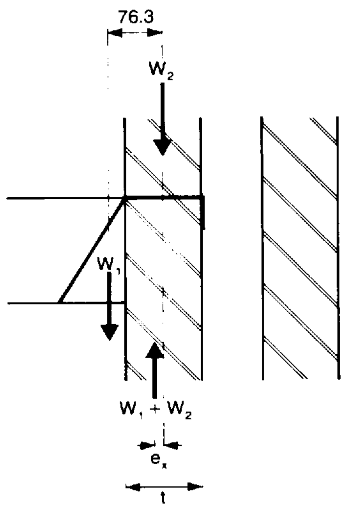

Figure 88 Wall 1– 2nd floor eccentricity.

32.2.1 ***Design vertical resistance of wall***
$=\dfrac{\beta t f_k}{\gamma_m}=\dfrac{0.65\times102.5\,f_k}{3.5}$
$=19.0\,f_k$
Design vertical load$=W_1+W_2=5.94+22.65$
$=28.59$ kN/m
Therefore, minimum f_k required$=\dfrac{28.59}{19.0}$
$=1.50$ N/mm^2
(b) Loading for maximum axial load:
$W_1=5.94$ kN/m (as for case (a))
$W_2=1.4\,G_{k2}+1.6\,Q_{k2}$
$=1.4\,(22.4-1.5)+1.6\,(4.8-2.4)$
$=33.1$ kN/m
Taking moments about the centre line:
$5.94\times76.3=(33.1+5.94)\,e_x$
Therefore, $e_x=\dfrac{453.2}{39.04}=11.6$ mm
$=0.11\,t.$
The slenderness ratio (as for case (a))$=17.5$.
Therefore, by interpolation, from code Table 7, β, =0.71.

32.2.1 ***Design vertical resistance of wall***
$=\dfrac{\beta t f_k}{\gamma_m}=\dfrac{0.71\times102.5\,f_k}{3.5}$
$=20.8\,f_k$
Design vertical load$=W_1+W_2=5.94+33.1$
$=39.04$ kN/m
Therefore, minimum f_k required$=\dfrac{39.04\times10^3}{20.8\times10^3}$
$=1.88$ N/mm^2

Section z-z (5200 mm long) 1st floor level

Characteristic loading

		G_k (dead load)	Q_k (imposed load)
rc beam G_k	$= \frac{24.2 \times 1.0}{5.2}$	$= 4.7$ kN/m	
Q_k	$= \frac{5.2 \times 1.0}{5.2}$		$= 1.0$ kN/m
2nd floor G_k	= as section x-x	$= 1.5$ kN/m	
Q_k	= as section x-x		$= 2.4$ kN/m
1st floor G_k	= as 2nd floor	$= 1.5$ kN/m	
Q_k	= as 2nd floor		$= 2.4$ kN/m
wall self-weight	$= 5.0 \times 0.5 \times (3.96 + 2.6)$	$= 16.4$ kN/m	
		24.1 kN/m	5.8 kN/m

Design loading at z-z

$1.4\,G_k = 1.4 \times 24.1 = 33.74$ kN/m

$1.6\,Q_k = 1.6 \times 5.8 = 9.28$ kN/m

43.02 kN/m

28. ***Slenderness considerations***

28.1 Slenderness ratio $= \frac{2400}{137}$

$= 17.5$, as before.

31. ***Eccentricity at right angles to wall, see Figure 89***

As above, the floor loading is applied 25 mm from the face of the wall.
The axial load is now greater than previously considered. Therefore, it is only necessary to consider the load case with the maximum axial load.

Loading for maximum axial load:

$W_1 = 5.94$ kN/m, as for section y-y

$W_2 = 1.4\,G_{k2} + 1.6\,Q_{k2}$

$= 1.4\,(24.1 - 1.5) + 1.6\,(5.8 - 2.4)$

$= 37.08$ kN/m

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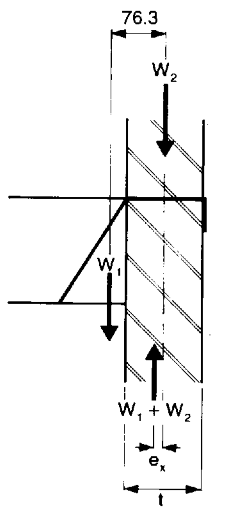

Figure 89 Wall 1 – 1st floor eccentricity.

Taking moments about wall centre line:

$5.94 \times 76.3 = (37.08 + 5.94)\,e_x$.

Therefore, $e_x = \frac{453.2}{43.02} = 10.5$ mm

$= 0.1\,t$.

The slenderness ratio $= 17.5$.

Therefore, by interpolation, from code Table 7, $\beta_, = 0.72$.

32.2.1 ***Design vertical resistance of wall***

$= \frac{\beta t f_k}{\gamma_m} = \frac{0.72 \times 102.5\, f_k}{3.5}$

$= 21.1\, f_k$

Design vertical load $= W_1 + W_2 = 5.94 + 37.08$

$= 43.02$ kN/m.

Therefore, minimum f_k required $= \frac{43.02 \times 10^3}{21.1 \times 10^3}$

$= 2.04$ N/mm^2.

Thus, section x-x is the critical section, and the minimum characteristic compressive strength of brickwork, f_k, required is 2.32 N/mm².
From code Figure 1, class 2 bricks in any mortar designation may be used.

In the above design at section z-z, the capacity reduction factor, β, is based upon the design eccentricity at 0.4h below z-z (see Chapter 2, page 37). This is conservative as the beam load would be spread over a greater length of wall at this level. Appendix B to the code can be used to derive the capacity reduction factor, β, at z-z if required as follows:
At z-z, $e_x = 10.5$ mm, additional eccentricity, $e_a, = 0$ (see Chapter 2, page 37).
Therefore, design eccentricity, $e_m, = 10.5$ mm.
From equation 4, Appendix B to the code, capacity reduction factor

$$\beta = 1.1\ (1 - \frac{2 \times 10.5)}{102.5} = 0.87$$

32.2.1 ***Design vertical resistance of wall***

$$= \frac{\beta t f_k}{\gamma_m} = \frac{0.87 \times 102.5\ f_k}{3.5}$$

$= 25.5\ f_k$
Design vertical load, as before,
$= 43.02$ kN/m.

Therefore, minimum f_k required $= \frac{43.02 \times 10^3}{25.5 \times 10^3}$
$= 1.69$ N/mm², compared to 2.04 N/mm² when using code Table 7 to obtain β.

Wall 2, 102.5 mm thick brickwork
An elevation of the wall is shown in Figure 85. The critical sections for design are sections x-x, y-y and between ground and first floor.

Section x-x (480 mm long) under padstone
Characteristic loading

	G_k (dead load)	Q_k (imposed load)
rc beam (as wall 1) $G_k = 14.9 \times 3.25$	$= 48.5$ kN	
$Q_k = 3.2 \times 3.25$		$= 10.5$ kN
2nd floor $G_k = 0.9 \times 1.8 \times 0.5 \times 4.25 \times 0.5 =$	1.7 kN	
$Q_k = 1.5 \times 1.8 \times 0.5 \times 4.25 \times 0.5$		$= 2.9$ kN
	50.2 kN	13.4 kN

Design loading at x-x
$1.4\ G_k = 1.4 \times 50.2 = 70.3$ kN
$1.6\ Q_k = 1.6 \times 13.4 = 21.4$ kN
91.7 kN
Stress due to design load

$$= \frac{91.7 \times 10^3}{102.5 \times 480} = 1.87 \text{ N/mm}^2.$$

34. The padstone bearing conforms to bearing type 1 (code Table 4), and as the wall is narrow, ie, its thickness is equal to the width of a standard format brick, f_k can be multiplied by 1.15.

23.1.2 Therefore, the design strength $= \frac{1.25\ (1.15\ f_k)}{\gamma_m}$
Assuming $\gamma_m = 3.5$,
$\frac{1.25\ (1.15\ f_k)}{3.5}$ must be greater than 1.87 N/mm².
Therefore, minimum f_k required = 4.55 N/mm².

Parallel calculations
If the rc beam had been located at the end of the wall, the padstone (spreader) would have been of bearing type 3 as in code Figure 4(c).

Maximum stress under spreader due to design loads
(See Chapter 3, page 62, for discussion of the following methods of analysis)

(a) Using Timoshenko's analysis for the bending of bars on elastic foundations (Strength of materials – Part 2):
Assume design load, P, $= 91.7$ kN acts at end of spreader, see Figure 90.
Select length of spreader as 1000 mm.
Moment of inertia of spreader, I_z,

$$= \frac{0.102 \times 0.225^3}{12} = 9.7 \times 10^{-5} \text{ m}^3.$$

Assume modulus of elasticity for concrete, E_c, $=25 \times 10^6$ kN/m².
Assume modulus of elasticity for brickwork, E_b, $=900 f_k$ $=10 \times 10^6$ kN/m²

Modulus of foundation, k, $= \frac{A\,S\,E_b}{L}$

$= \frac{0.102 \times 1 \times 10 \times 10^6}{2.5} = 4.1 \times 10^5$

where:
A = area under unit length of spreader.
S = unit deflection
L = height of wall.
k may be defined as a constant denoting the reaction per unit length, when the deflection is equal to unity.

Deflection beneath the load $= \frac{P}{2\beta^3 E_c I_z}$

Where constant $\beta^* = \left(\frac{k}{4 E_c I_z}\right)^{\frac{1}{4}} = \left(\frac{4.1 \times 10^5}{4 \times 25 \times 10^6 \times 9.7 \times 10^{-5}}\right)^{\frac{1}{4}}$
$=2.55$.
Therefore, deflection beneath the load

$= \frac{91.7}{2 \times 2.55^3 \times 25 \times 10^6 \times 9.7 \times 10^{-5}}$
$=0.0011$ m.
Reaction per unit length $=$ k $\times$ deflection $= 4.1 \times 10^5 \times 0.0011$
$=451$ kN/m.

Therefore, maximum stress $= \frac{451 \times 10^3}{1000 \times 102}$

$=4.42$ N/mm².

**Note β must not be confused with the capacity reduction factor in the code.*

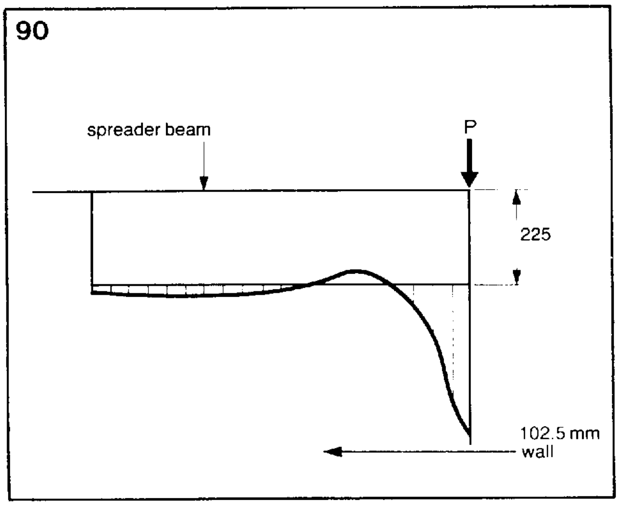

Figure 90 Stress distribution under spreader beam, after Timoshenko.

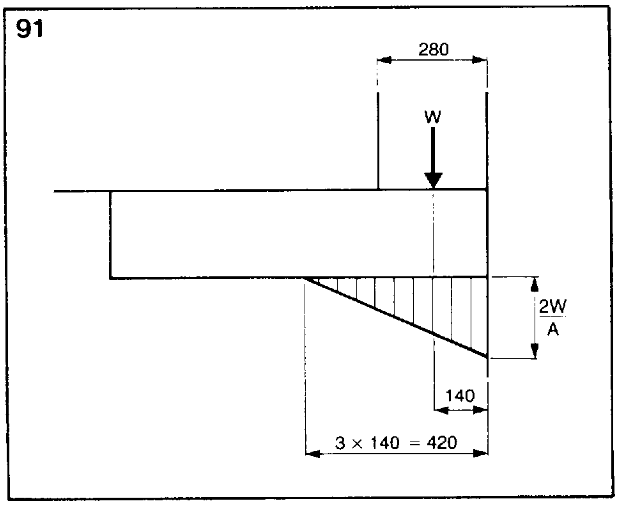

Figure 91 Stress distribution under spreader beam, triangular stress block.

(b) Using triangular stress block:

Assume design load, W, $=91.7$ kN, acts at centre line of wall, see Figure 91.
Length of stress block $= 3 \times 140 = 420$ mm.
(padstone need not exceed 420 mm in length; if it does, theoretically tension occurs under the padstone.)

Therefore, maximum stress $= \frac{2W}{A}$

$= \frac{2 \times 91.7 \times 10^3}{420 \times 102} = 4.28$ N/mm².

Section y-y (1215 mm long) 0.4h below underside of rc beam
Characteristic loading

		G_k (dead load)	Q_k (imposed load)
rc beam G_k	$=\frac{50.2}{1.215}$	$=41.3$ kN/m	
Q_k	$=\frac{13.4}{1.215}$		$=11.0$ kN/m
wall self-weight	$=2.75\times 0.96=$	2.6 kN/m	
		43.9 kN/m	11.0 kN/m

Design loading at y-y
$1.4\ G_k=1.4\times 43.9=61.5$ kN/m

$1.6\ Q_k=1.6\times 11.0=17.6$ kN/m
79.1 kN/m

28. ***Slenderness considerations***
28.2.2.1 Simple resistance to lateral movement is provided by one intermediate strap as shown in code Appendix C, Figure 21, between the rc beam and the external wall.
28.3.1.1 Effective height, h_{ef} $=2400$ mm.
28.4 Effective thickness $=$ actual thickness $=102.5$ mm
28.1 Therefore, slenderness ratio $=\frac{2400}{102.5}=23.4$

31. ***Eccentricity at right angles to the wall***
The wall is symmetrically loaded, therefore $e_x=0$
By interpolation from code Table 7, $\beta,=0.56$

23.1.2 ***Design vertical resistance of wall,*** (*including narrow brick wall factor*)

$$\frac{\beta\, t(1.15f_k)}{\gamma_m}=\frac{0.56\times 102.5\times 1.15\, f_k}{3.5}$$
$=18.9\ f_k$
Design vertical load $=79.1$ kN/m.

Therefore, minimum f_k required $=\frac{79.1\times 10^3}{18.9\times 10^3}$
$=4.19\ \text{N/mm}^2$

Ground to first floor (2700 mm long)
For the cavity wall section, the vertical load is carried on the inner leaf only.
Characteristic loading

		G_k (dead load)	Q_k (imposed load)
rc beam G_k	$=\frac{50.2}{2.70}$	$=18.6$ kN/m	
Q_k	$=\frac{13.4}{2.70}$		$=5.0$ kN/m
First floor G_k	$=0.9\times 3.25+0.9\times 1.8\times 0.5\times 4.25\times 0.5\times\frac{1}{2.6}=$	3.6 kN/m	
Q_k	$=1.5\times 3.25+1.5\times 1.8\times 0.5\times 4.25\times 0.5\times\frac{1}{2.6}$		$=6.0$ kN/m
Wall self-weight (full height of wall)	$=2.75\times 5.2$	$=14.3$ kN/m	
		36.5 kN/m	11.0 kN/m

Design loading
$1.4\ G_k=1.4\times 36.5=51.1$ kN/m
$1.6\ Q_k=1.6\times 11.0=17.6$ kN/m
68.7 kN/m

28. ***Slenderness considerations***
28.2.2.2 Timber joists spanning onto the wall from both sides provide enhanced resistance to lateral movement.
28.3.1.1 Effective height, h_{ef}, $=0.75\times 2400=1800$ mm
28.4 Effective thickness $=102.5$ mm, as at y-y.
28.1 Therefore, slenderness ratio $=\frac{1800}{102.5}=17.6$

31. ***Eccentricity at right angles to the wall***

$e_x = 0$, as at y-y.

The design loading and slenderness ratio at this level are lower than at y-y, and the eccentricity is the same. Therefore, by inspection, the required f_k is less than the 4.19 N/mm² at y-y.

The wall will be designed for the maximum value of f_k required, ie, 4.19 N/mm².

Using standard format bricks, from code Figure 1, class 2 bricks in any of the mortar designations (i) to (iii) may be used.

Wall 3, 100 mm thick solid concrete blockwork

The critical section for design is from ground to first floor (see Figure 85).

Characteristic loading

	G_k (dead load)	Q_k (imposed load)
Floors $G_k = 2 \times 0.9 \times 3.25 + 2 \times 0.9 \times 1.8 \times 0.5 \times 4.25 \times 0.5 \times \frac{1}{2.6}$	= 7.2 kN/m	
$Q_k = 2 \times 1.5 \times 3.25 + 2 \times 1.5 \times 1.8 \times 0.5 \times 4.25 \times 0.5 \times \frac{1}{2.6}$		= 12.0 kN/m
wall self-weight = $1.4 \times 2.6 \times 3$	= 10.9 kN/m	
	18.1 kN/m	12.0 kN/m

Design loading

$1.4\,G_k = 1.4 \times 18.1 = 25.3$ kN/m

$1.6\,Q_k = 1.6 \times 12.0 = 19.2$ kN/m

44.5 kN/m

28.

28.2.2.2 ***Slenderness considerations***

Timber joists spanning onto the wall from both sides provide enhanced resistance to lateral movement.

28.3.1.1 Effective height, h_{ef}, $= 0.75 \times 2400 = 1800$ mm

28.4 Effective thickness = actual thickness = 100 mm

28.1 Therefore, slenderness ratio $= \frac{1800}{100} = 18$

31. ***Eccentricity at right angles to the wall***

The wall is symmetrically loaded, therefore $e_x = 0$

From code Table 7, $\beta = 0.77$

32.2.1 ***Design vertical resistance of wall***

$$= \frac{\beta t f_k}{\gamma_m} = \frac{0.77 \times 100 \times f_k}{3.5} = 22.0\,f_k$$

Design vertical load = 44.5 kN/m

Therefore, f_k required $= \frac{44.5 \times 10^3}{22.0 \times 10^3} = 2.02$ N/mm²

Using 215 mm high × 440 mm long solid concrete blocks (ratio of height to least horizontal dimension = 215:100 = 2.15), from code Table 2(d), a block with a unit compressive strength of 2.8 N/mm² in any mortar designation may be used.

Wall 4, 280 mm cavity external wall, with 102.5 mm facing brick outer leaf & 100 mm lightweight solid concrete blockwork inner leaf

The critical sections for design of the inner, loadbearing, leaf are sections x-x, y-y, and z-z (see Figure 86).

Section x-x (730 mm long) under padstone

Characteristic loading

	G_k (dead load)	Q_k (imposed load)
rc beam $G_k = 14.9 \times 3.25 \times 0.5$	= 24.2 kN	
$Q_k = 3.2 \times 3.25 \times 0.5$		= 5.2 kN
wall self-weight = say, $1.5 \times 0.73 \times 3.0$	= 2.5 kN	
	26.7 kN	5.2 kN

Design loading at x-x

$1.4\,G_k = 1.4 \times 26.7 = 37.4$ kN

$1.6\,Q_k = 1.6 \times 5.2 = 8.3$ kN

45.7 kN

Stress due to design load $= \frac{45.7 \times 10^3}{100 \times 730} = 0.63$ N/mm².

Length of padstone, 730 mm, exceeds 6 × wall thickness (code Figure 4) and, therefore, no increased local stress is permissible.

Design strength of wall $= \frac{f_k}{\gamma_m}$, which must be greater than 0.63 N/mm².

Therefore, minimum f_k required $= 0.63 \times 3.5$
$= 2.21$ N/mm², assuming $\gamma_m = 3.5$

Section y-y (2200 mm long) 0.4h below underside of rc beam

Characteristic loading

		G_k (dead load)	Q_k (imposed load)
rc beam G_k	$= \frac{24.2}{2.2}$	= 11.0 kN/m	
Q_k	$= \frac{5.2}{2.2}$		= 2.4 kN/m
wall self-weight = say, 1.15×3.5		= 4.0 kN/m	
		15.0 kN/m	2.4 kN/m

Design loading at y-y

$1.4\,G_k = 1.4 \times 15.0 = 21.0$ kN/m
$1.6\,Q_k = 1.6 \times 2.4 = 3.8$ kN/m
24.8 kN/m

28. ***Slenderness considerations***

28.2.2.1 Simple resistance to lateral movement is provided as for wall 1 (see page 97).
28.3.1.1 Effective height, h_{ef}, = 2400 mm.
28.4 Effective thickness, t_{ef}, $= 0.67\,(100 + 102.5)$
$= 136$ mm.

28.1 Therefore, slenderness ratio $= \frac{2400}{136}$
$= 17.6$.

31. ***Eccentricity at right angles to the wall***

The reinforced concrete beam is of sufficient stiffness and short enough span to enable its reaction on the wall to be considered as an axial load. As the staircase is adjacent to one side of the beam, the only other load to be considered is the wall self-weight which is axial.
Therefore the eccentricity, e_x, = 0.
By interpolation from code Table 7, β, = 0.78.

32.2.1 ***Design vertical resistance of wall***

$= \frac{\beta t f_k}{\gamma_m} = \frac{0.78 \times 100 f_k}{3.5}$
$= 22.3 f_k$.
Design vertical load = 24.8 kN/m
Therefore, minimum f_k required $= \frac{24.8 \times 10^3}{22.3 \times 10^3}$
$= 1.11$ N/mm².

Section z-z, ground to first floor

Characteristic loading

		G_k (dead load)	Q_k (imposed load)
rc beam G_k	$= \frac{24.2}{3.08}$	= 7.9 kN/m	
Q_k	$= \frac{5.2}{3.08}$		= 1.7 kN/m
1st floor G_k	$= 0.9 \times 3.25 \times 0.5$	= 1.5 kN/m	
Q_k	$= 1.5 \times 3.25 \times 0.5$		= 2.4 kN/m
wall self-weight = say, 1.15×7		= 8.1 kN/m	
		17.5 kN/m	4.1 kN/m

Design loading

$1.4\,G_k = 1.4 \times 17.5 = 24.5$ kN/m
$1.6\,Q_k = 1.6 \times 4.1 = 6.6$ kN/m
31.1 kN/m

28. ***Slenderness considerations***

28.1 Slenderness ratio = 17.6, as section y-y.

31. ***Eccentricity at right angles to the wall, see Figure 92.***

It was explained in chapter 3, and has been shown in this example (page 98), that the load combination giving the maximum design load is usually the critical one. Therefore, only this case will be considered.

Loading

$W_1 = 1.4\,G_{k1} = 1.4 \times 1.5 = 2.1$ kN/m
$+ 1.6\,Q_{k1} = 1.6 \times 2.4 = \underline{3.8}$ kN/m
5.9 kN/m
$W_2 = 31.1 - 5.9 = \underline{25.2}$ kN/m
$W_1 + W_2 = \underline{31.1}$ kN/m

Taking moments about the wall centre line:

$5.9 \times 75 = 31.1 e_x$

Therefore $e_x = \dfrac{442.5}{31.1} = 14.2$ mm

$= 0.14\,t.$

By interpolation from code Table 7, $\beta, = 0.66$.

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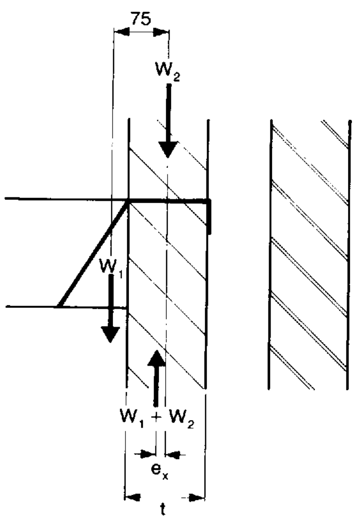

Figure 92 Wall 4 – 1st floor eccentricity.

32.2.1 ***Design vertical resistance of wall***

$= \dfrac{\beta\, t\, f_k}{\gamma_m} = \dfrac{0.59 \times 100 \times f_k}{3.5}$

$= 18.9\, f_k.$

Design vertical load $= 31.1$ kN/m.

Therefore, minimum f_k required $= \dfrac{31.1 \times 10^3}{18.9 \times 10^3}$

$= 1.65$ N/mm^2.

The wall will be designed for the maximum value of f_k required, ie, 2.21 N/mm^2. Using 215 mm high $\times$ 440 mm long solid concrete blocks (ratio of height to least horizontal dimension 215 : 100 = 2.15), from code Table 2(d), blocks with a unit compressive strength of 2.8 N/mm^2 in any mortar designation may be used.

Wall 5, 280 mm cavity external wall with 102.5 mm facing brick outer leaf and 100 mm lightweight solid concrete blockwork inner leaf

The critical section for the design of the inner, loadbearing, leaf is section x-x (see Figure 87).

Characteristic loading

	G_k (dead load)	Q_k (imposed load)
Roof lintols a and b:		
roof $G_k = 1.1 \times 6.1 \times 0.5$	= 3.4 kN/m	
$Q_k = 0.75 \times 6.1 \times 0.5$		= 2.3 kN/m
self-weight of lintol, say	= 1.0 kN/m	
	4.4 kN/m	2.3 kN/m
Floor level lintols c:		
self-weight of wall over $= 1.15 \times 1.1$	= 1.3 kN/m	
glazing $= 0.5 \times 1.25$	= 0.6 kN/m	
self-weight of lintol, say	= 1.0 kN/m	
	2.9 kN/m	

Floor level lintol d:

Self-weight of wall over $=1.15\times3.7$	$=$	4.3 kN/m
self-weight of lintol, say	$=$	1.0 kN/m
		5.3 kN/m

Total characteristic load at section x-x

lintols a and b	$G_k=4.4\times3.825\times0.5$	$=8.4$ kN	
	$Q_k=2.3\times3.825\times0.5$		$=4.4$ kN
lintols c	$G_k=2.9\times2.025\times0.5\times2$	$=5.9$ kN	
lintol d	$G_k=5.3\times1.8\times0.5$	$=4.8$ kN	
roof	$G_k=1.1\times6.1\times0.5\times0.675$	$=2.3$ kN	
	$Q_k=0.75\times6.1\times0.5\times0.675$		$=1.5$ kN
wall self-weight	$G_k=1.15\times0.675\times2.6\times3$	$=6.1$ kN	
		27.5 kN	5.9 kN

28. ***Slenderness considerations***

28.2.3.2 It is assumed that the central loadbearing partition wall at right angles to wall 5 is properly bonded to the inner, loadbearing, leaf of wall 5, thus providing a vertical support with enhanced resistance to lateral movement. As wall 5 is not an isolated vertical loadbearing member, it will be considered as a wall for design purposes.

28.3.2 The section of wall under consideration is x-x in Figure 87, therefore the slenderness ratio will be based on effective length.
The effective length of the wall, $h_{ef}=2.0\times400=800$ mm.
Effective thickness, t_{ef}, $=0.67\ (100+102.5) \quad =136$ mm.

Therefore, slenderness ratio $=\dfrac{800}{136}=5.9$

31. ***Eccentricity at right angles to the wall***
No eccentricity of loading need be considered in this direction as the roof and lintol loads are axial.

30. ***Eccentricity in the plane of the wall***
Whilst wind forces have been assumed to produce negligible eccentricity on the walls in this direction, an eccentric load is introduced by the loads from the concrete lintols spanning onto wall 5.

Design loading at section x-x, see figure 93

$W_1=2.9\times2.025\times0.5\times1.4= \ 4.1$ kN
$W_2=5.3\times1.8\times0.5\times1.4 \ = \ 6.7$ kN
$\qquad\qquad\qquad\qquad 10.8$ kN

Total design vertical load:
$1.4\,G_k=1.4\times27.5 \quad =38.5$ kN
$1.6\,Q_k=1.6\times5.9 \quad = \ 9.4$ kN
$\qquad\qquad\qquad 47.9$ kN

Therefore, $W_3=47.9-10.8=37.1$ kN.
It is assumed that the lintols butt over the centre line of the wall, or are continuous, and the positions of W_1 and W_2 have been assessed from code clause 31.

Taking moments about the centre line of the wall:

$$47.9\,e_x=(6.7-4.1)\times\frac{675}{3}$$

Therefore, $e_x=12.2$ mm $=0.018\,L$

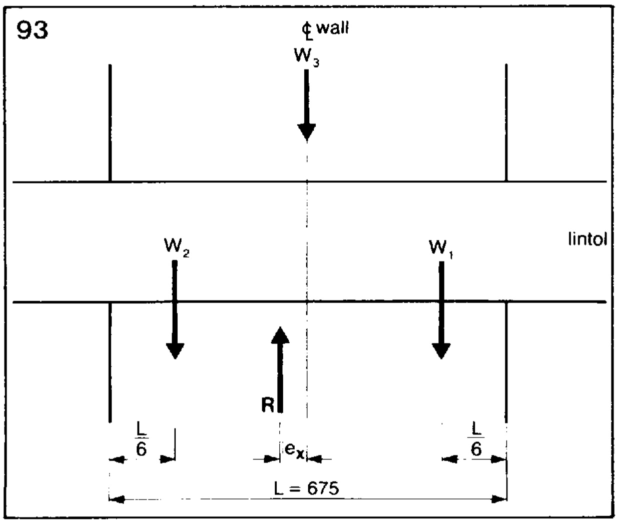

Figure 93 Wall 5 – eccentricity at x – x.

$$\text{Design stress} = \frac{W}{A} \pm \frac{M}{Z} = \frac{47.9 \times 10^3}{675 \times 100} \pm \frac{47.9 \times 10^3 \times 12.2 \times 6}{100 \times 675^2}$$

$= 0.710 \pm 0.077\ \text{N/mm}^2$

$= 0.79$ or $0.63\ \text{N/mm}^2$

Design load per unit length $= 79$ kN/m

Therefore, by interpolation from code Table 7, $\beta, = 1.0$

32.2.1 ***Design vertical resistance of wall***

Horizontal loaded cross-sectional area of wall, A,

$= \frac{675 \times 100}{10^6} = 0.068\ \text{m}^2$

Therefore, the characteristic compressive strength of the masonry must be multiplied by the factor:

$(0.7 + 1.5A) = (0.7 + 1.5 \times 0.068) = 0.8$,

and the design vertical resistance of wall 5

$= \frac{\beta t 0.8 f_k}{\gamma_m} = \frac{1.0 \times 100 \times 0.8\ f_k}{3.5}$

$= 22.9\ f_k$, assuming $\gamma_m = 3.5$

Therefore, minimum f_k required $= \frac{79 \times 10^3}{22.9 \times 10^3}$

$= 3.45\ \text{N/mm}^2$

Using 215 mm high $\times$ 440 mm long solid concrete blocks (height to least horizontal dimensional ratio 215:100 = 2.15), from code Table 2(d), blocks with a unit compressive strength of 3.5 N/mm² in any mortar designation may be used.

Design of walls for lateral loading

The worst wall from a lateral load point of view is the gable wall as it has no precompression from the roof.

The gable wall is tied back to the trussed rafter roof structure at eaves and ceiling levels in accordance with code Appendix C and the roof is adequately braced to transmit the forces from these ties to the longitudinal walls of the structure.

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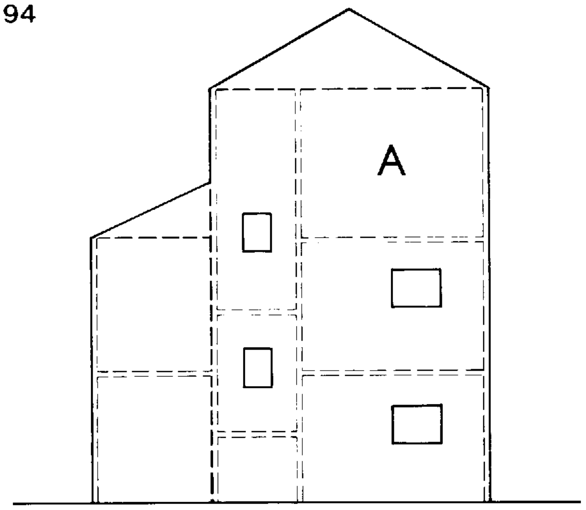

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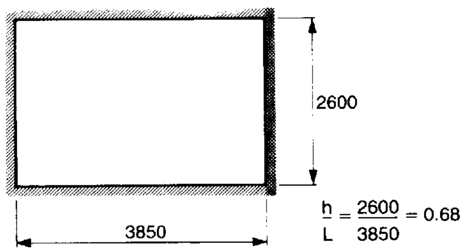

Figure 95 Edge support conditions – Wall A.

Figure 94 Elevation of gable wall.

Consider Wall A, Figure 94:

The wall is simply supported top and bottom. Where the masonry is continuous around the corner of the building, a continuous edge may be assumed. Whilst the wall is continuous past an internal partition at the other end, it will be considered as simply supported as there is a window opening close to the partition over the lower part of the wall (Figure 95).

36.3 *Limiting dimensions*

For a panel supported on four sides with less than three sides continuous, the height $\times$ length of the panel must be less than or equal to 2025 t_{ef}.

$t_{ef} = 0.67\ (102.5 + 100) \quad = 136.0$ mm

Therefore, $2025 \times 136.0^2 = 37.5 \times 10^6\ \text{mm}^2$

$h \times L = 2600 \times 3850 \quad = 10.0 \times 10^6\ \text{mm}^2$

this is less than $2025 t_{ef}^2$ and therefore satisfactory.

The maximum wall dimension must not exceed $50 t_{ef}$

$= 50 \times 136.0 = 6800$ mm.

Wall length = 3850 mm, therefore the wall satisfies the limiting dimensions clause.

Wind loading

Dynamic wind pressure, q, =0.537 kN/m (see page 96).

Pressure coefficients

Assume length of terrace is 28.0 m and average height is 7.8 m.

Building dimension ratios:

$\frac{\text{height}}{\text{width}}=\frac{7.8}{8.6}=0.91$, $\frac{\text{length}}{\text{width}}=\frac{28.0}{8.6}=3.26$.

Therefore, from CP 3: Chapter V: Part 2, Table 7,

maximum $C_{pe}=\pm 0.7$

maximum $C_{pi}=-0.3$ or $+0.2$.

Therefore, maximum $C_{pe}-C_{pi}=+0.7-(-0.3) \quad =+1.0$

local $C_{pe}=-1.1$

Therefore, maximum local $C_{pe}-C_{pi}=-1.1-0.2 \quad =-1.3$

The local pressure acts over a quarter of the width of the building $=0.25\times 8.6=2.15$ m.

For the purposes of this example, it will be assumed that the local pressure (suction) acts over the full width of the wall being designed.

Therefore, characteristic wind load, W_k, $=1.3\times 0.537=0.7$ kN/m².

Design wind load, $\gamma_f W_k$, $=1.4\times 0.7 \quad =0.98$ kN/m².

22(b) γ_f is taken as 1.4 as the wall contributes to the overall stability of the structure.

24. ***Characteristic flexural strength***

Outer leaf

Assume the outer leaf is of clay bricks, with a water absorption of between 7% and 12%, in mortar designation (iii). Thus, from code Table 3, the characteristic flexural strength, f_{kx}, of the brickwork is 0.4 N/mm² and 1.1 N/mm² in the two orthogonal directions.

36.4.5 The vertical load due to the self-weight of the top half and gable section of the outer leaf acts so as to increase its flexural strength in the parallel direction.

Design vertical dead load due to self-weight of the top half and gable of outer leaf $=0.9\times 2.3\times 2.25=4.66$ kN/m (note γ_f is taken as 0.9).

Stress due to design vertical load

$$=\frac{4.66\times 10^3}{102.5\times 1000}=0.045 \text{ N/mm}^2$$

Therefore, the modified orthogonal ratio, μ,

$$=\frac{0.4+0.045\times 3.5}{1.1}=0.51$$

Inner leaf

The inner leaf is of lightweight solid concrete blockwork in mortar designation (iii). The blocks have a unit compressive strength of 3.5 N/mm². Thus, from code Table 3, the characteristic flexural strengths are 0.25 N/mm² and 0.45 N/mm². As for the outer leaf, the self-weight of the wall acts to modify the orthogonal ratio.

Design vertical dead load due to self-weight of top half and gable of inner leaf $=0.9\times 2.3\times 1.15=2.38$ kN/m

Stress due to design vertical load

$$=\frac{2.38\times 10^3}{100\times 1000}=0.024 \text{ N/mm}^2$$

Therefore the modified orthogonal ratio, μ,

$$=\frac{0.25+0.024\times 3.5}{0.45}=0.74$$

Because the orthogonal ratios of the leaves differ, it is necessary to calculate the design moments of resistance of each leaf, in order to apportion the applied horizontal force between them.

36.4.5 Design moments of resistance:

The design moment of resistance of the outer leaf (o) in the perpendicular direction, see page 65.

$$=\frac{f_{kxo}}{\gamma_m}Z_o=\frac{1.1}{3.5}\times\frac{1000\times 102.5^2}{6}\times\frac{1}{10^6}$$

$=0.55$ kNm/m.

The design moment of resistance of the inner leaf (I) in the perpendicular direction, see page 65.

$$=\frac{f_{kxI}}{\gamma_m}Z_I=\frac{0.45}{3.5}\times\frac{1000\times 100^2}{6}\times\frac{1}{10^6}$$

$=0.21$ kNm/m

Thus, the load taken by the outer leaf

$=\frac{0.55}{0.55+0.21}\times 0.98=0.71$ kN/m²

and by the inner leaf

$=0.98-0.71 \quad =0.27$ kN/m²

Bending moment coefficients:

Outer leaf, from code Table 9F for $\frac{h}{L}=0.68$ and $\mu=0.51$

$=0.033$

Inner leaf, from code Table 9F for $\frac{h}{L}=0.68$ and $\mu=0.74$

$=0.027$

36.4.2 Design bending moments:

Design bending moment on outer leaf in the perpendicular direction

$=\alpha_o W_{ko}\gamma_f L^2=0.033\times 0.71\times 3.85^2$ (Note $0.71=W_{ko}\gamma_f$)

$=0.35$ kNm/m

This is less than the design moment of resistance of the outer leaf and is therefore satisfactory.

Design bending moment on the inner leaf in the perpendicular direction

$=\alpha_1 W_{kI}\gamma_f L^2=0.027\times 0.27\times 3.85^2$ (note $0.27=W_{kI}\gamma_f$)

$=0.108$ kNm/m

This is less than the design moment of resistance of the inner leaf and is therefore satisfactory.

The remaining gable walls are satisfactory either because of their short span or, for the lower panels, the vertical load from above. The walls to the back elevation of the building contain substantial openings, and whilst the code deals in general terms with laterally loaded walls with openings in Appendix D, it gives little guidance on the design of these walls, except for recommending the sub-division of the wall into smaller sub-panels.

EXAMPLE 3

Design of Piers

Consider a single storey warehouse building consisting of a sheeted roof supported by shallow pitched steel trusses which are in turn supported on loadbearing masonry piers. The building is 4.0 m high to eaves level and the trusses, which are at 3.6 m centres, span 20 m. The masonry piers are in clay brickwork and are bonded into the brickwork inner leaf of a cavity wall which forms the external wall of the building.

The roof deck acts as a deep girder to transfer the wind forces on the side walls back to the gable walls.

Design of a typical pier:

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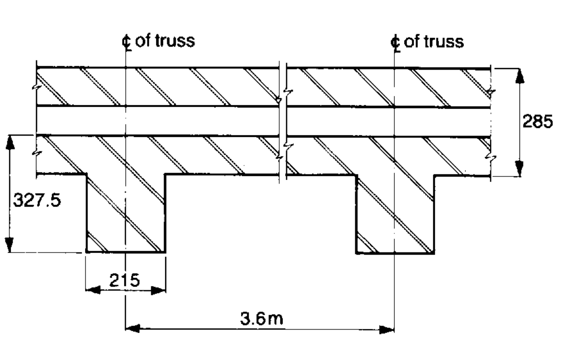

Figure 96 Plan detail – wall with piers.

Characteristic loading

Roof load:

Dead load $G_k=1.75$ kN/m²

Imposed load $Q_k=0.75$ kN/m²

Wind pressure $W_k=0.5$ kN/m²

Pier design

Assume a pier size of 327.5 mm by 215 mm, as shown in Figure 96.

Design for vertical load
Loading case (a): design load per pier

$$\text{Design dead load} = 1.4\ G_k = 1.4 \times 1.75 \times \frac{20}{2} \times 3.6 = 88.2\ \text{kN}$$

$$\text{Design imposed load} = 1.6\ Q_k = 1.6 \times 0.75 \times \frac{20}{2} \times 3.6 = 43.2\ \text{kN}$$

$$131.4\ \text{kN}$$

28.3.1.4 $\dfrac{\text{Thickness of pier}}{\text{Thickness of inner leaf of cavity wall}} = \dfrac{327.5}{102.5} = 3.2$

This is greater than 1.5, and therefore the pier should be treated as a column in the plane at right angles to the wall.

28. ***Slenderness considerations***

The slenderness ratio of the pier need be considered only in one plane, as the wall restrains the pier against buckling in the plane of the wall. Had the thickness ratio been less than 1.5, the pier could have been considered as a wall for effective height purposes.

28.3.1.2 Effective height of pier $= 4000$ mm

28.4.1 Effective thickness of pier $= 327.5$ mm

28.1 Therefore, slenderness ratio $= \dfrac{4000}{327.5}$

$= 12.2$

31. ***Eccentricity at right angles to the wall***

The truss fixing to the padstone on top of the pier is arranged to provide axial loading on the pier.

32.2.1 ***Design vertical load resistance of pier***

For a slenderness ratio of 12.2 and axial loading, from code Table 7 the capacity reduction factor, $\beta, = 0.92$

27.3 Assuming normal categories of manufacturing and construction control, the partial safety factor for material strength, $\gamma_m, = 3.5$, from code Table 4.

Design vertical load resistance

$$= \frac{\beta\ b\ t\ f_k}{\gamma_m} = \frac{0.92 \times 215 \times 327.5\ f_k}{3.5}$$

$$= 18.5\ f_k \times 10^3\ \text{N}$$

23.1.1 If the pier is correctly bonded into the inner leaf of the cavity wall, it is unreasonable to apply the factor for columns of small plan area to the characteristic compressive strength, f_k, of the masonry. If, however, any doubt exists as to the efficiency of the bonding, the factor should be applied – as is done here, by way of example:

Area of pier $= 0.3275 \times 0.215 = 0.07\ \text{m}^2$.

Therefore, f_k should be multiplied by $0.7 + 1.5 \times 0.07 = 0.8$, and the design vertical load resistance becomes:

$18.5 \times 0.8\ f_k \times 10^3 = 14.8\ f_k \times 10^3$ N.

Therefore, equating design load and design resistance:

$$\text{minimum } f_k \text{ required} = \frac{131.4 \times 10^3}{14.8 \times 10^3}$$

$= 8.9\ \text{N/mm}^2$.

From code Figure 1, a class 4 brick in mortar designation (i) may be used.

Design for lateral load
Loading case (b)

Assume net design dead load, allowing for uplift, $(0.9\ G_k - 1.4\ W_k\ (\text{uplift})) = 1.125\ \text{kN/m}^2$.

$$\text{Design dead load on pier} = 1.125 \times \frac{20}{2} \times 3.6 = 40.5\ \text{kN}$$

Design wind load $= 1.4\ W_k = 1.4 \times 0.5 = 0.7\ \text{kN/m}^2$ (suction)

Design wind load on pier $= 0.7 \times 3.6 = 2.52$ kN/m,

assuming for simplicity that the cavity wall spans horizontally between piers.

36.4.2 Design moment on piers, assuming simple support top and bottom

$$= \frac{2.52 \times 4.0^2}{8} = 5.04\ \text{kNm}$$

If necessary, a moment at the base of the wall due to the self-weight of the pier may be used to reduce the design moment on the pier.

36.4.3 Design moment of resistance of pier:

In assessing the section modulus of a pier, the outstanding length of the flanges may be taken as 6 times the thickness of the inner leaf, see Figure 97.

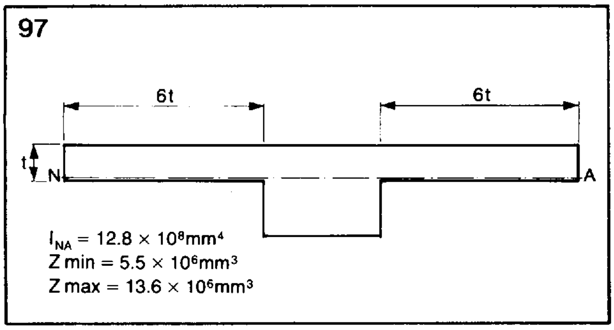

Figure 97 Section properties of pier.

Assuming that the bricks used have a water absorption greater than 12% and that mortar designation (i) is used the characteristic flexural strength of the masonry in the parallel direction is 0.4 N/mm² from code Table 3.

Stress due to design vertical load $= \dfrac{40.5 \times 10^3}{327.5 \times 215} = 0.58$ N/mm²

Therefore design moment of resistance of pier

$= \left(\dfrac{0.4}{3.5} + 0.58\right) \times \dfrac{5.5 \times 10^6}{10^6} = 3.82$ kNm, which is unsatisfactory.

Neither allowing for the self-weight of the top half of the pier, nor using a brick with a lower water absorption will improve the moment of resistance of the pier sufficiently, therefore its size must be increased.

Try a 327.5 mm square pier

$I = 18.0 \times 10^8$ mm⁴

$Z\ min = 8.03 \times 10^6$ mm³

Therefore design moment of resistance of pier

$= \left(\dfrac{0.4}{3.5} + 0.58\right) \times \dfrac{8.03 \times 10^6}{10^6} = 5.6$ kNm

This exceeds the design moment, 5.04 kNm, and is therefore satisfactory.

Because of the increase in pier size to resist wind loading, a reduced minimum characteristic compressive strength of brick, f_k, may be calculated if desired, using the load case giving the greatest value of f_k.

EXAMPLE 4

Laterally loaded wall panel

Consider a single-storey steel frame building on the outskirts of Manchester. The structure is clad with masonry in the form of a 265 mm thick cavity wall. The outer leaf is of a clay facing brick, and the inner leaf of a 100 mm thick lightweight aggregate block of 3.5 N/mm² unit compressive strength. Both leaves are built in mortar designation (iii) and pass outside the columns. The columns are at 4.0 m centres, and the panels are 3.0 m high to the underside of the independently supported clerestorey windows. The masonry is adequately tied to the steel columns. It is assumed that the frame is braced independently of the cladding for overall stability. Expansion joints are provided in the cladding at the corners of the structure and at every 12 m. Design the cladding to resist wind forces.

The critical panel is the end panel which will be subjected to local wind pressure.

Wind pressure (from CP 3: Chapter V, Part 2)

Basic wind speed for Manchester, say, 46 m/s.

Design wind speed factors, S_1 and $S_3 = 1.0$

$S_2 = 0.70$ (ground roughness category (3), class A, height of building = 5.0 m).

Therefore, design wind pressure, V_s, $= 46.0 \times 1.0 \times 0.70 \times 1.0$

$= 32.2$ m/s

and dynamic wind pressure, q, $= \dfrac{0.613 \times 32.2^2}{10^3}$

$= 0.64$ kN/m².

Assume $\dfrac{\text{height of building}}{\text{width}} < \dfrac{1}{2}$

and $\dfrac{\text{length of building}}{\text{width}} > \dfrac{3}{2}$ but < 4

and that there are no dominant openings,

the local coefficient of wind pressure, external, $C_{pe} = -1.0$

internal, $C_{pi} = +0.2$.

Therefore, total local coefficient of wind pressure

$= C_{pe} - C_{pi} = -1 - (+0.2) = -1.2$.

The characteristic local suction wind load, W_k,

$= 1.2 \times 0.64 = 0.77$ kN/m²

22(b) The design wind load $=\gamma_f W_k$. As the removal of the panel would not affect the stability of the building, γ_f may be taken as 1.2.
Design wind load $=1.2\times0.77=0.92$ kN/m²

24. ***Characteristic flexural strength***
From code Table 3, assuming the use of a clay brick with a water absorption of less than 7% for the outer leaf, the orthogonal characteristic flexural strengths, f_{kx}, are 0.5 N/mm² and 1.5 N/mm². The corresponding f_{kx} values for the inner leaf are 0.25 N/mm² and 0.45 N/mm². These values give an orthogonal ratio, μ, of $\frac{0.5}{1.5}=0.33$ for the outer leaf and $\frac{0.25}{0.45}=0.56$ for the inner leaf.

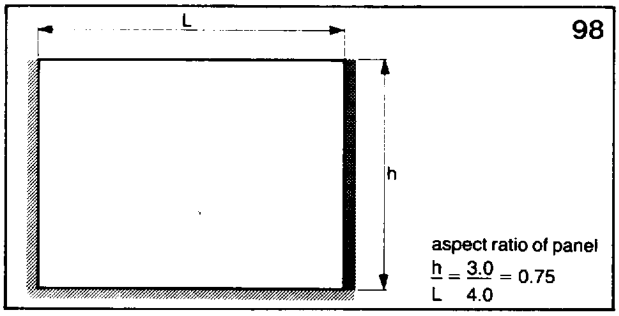

Figure 98 Panel edge support conditions.

The panel is considered simply supported at the corner of the building and continuity at its other end provides fixity. The bottom is considered simply supported and the top edge free. Thus, the panel is as shown in Figure 98.

36.3 *Limiting panel dimensions*
The panel is case (a) (2), ie, supported on three edges with less than two edges continuous. Therefore, height × length must not exceed 1350 t_{ef}^2, and the maximum panel dimension must not exceed 50 t_{ef}
$t_{ef}=0.67\,(102.5+100)=136$ mm.

$$\text{Therefore, } 1350\,t_{ef}^2 = \frac{1350\times136^2}{10^6} = 25.0 \text{ m}^2$$

and $50\,t_{ef}=50\times136 = 6800$ mm
Now, height × length $=3.0\times4.0=12.0$ m², which is less than 25.0 m² and is therefore satisfactory. The maximum panel dimension is 4000 mm, and this is less than 6800 and again satisfactory.

36.4.3 *Design moment of resistance of laterally loaded wall*

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

Section modulus, outer leaf

$$=\frac{1}{6}\times1000\times102.5^2=1.75\times10^6 \text{ mm}^3\text{/m}$$

Section modulus, inner leaf

$$=\frac{1}{6}\times1000\times100^2 = 1.67\times10^6 \text{ mm}^3\text{/m}$$

Therefore, design moment of resistance of outer leaf, using the f_{kx} value for the plane of failure perpendicular to bed joints, see Figure 1,

$$=\frac{1.5\times1.75\times10^6}{3.5}=0.75 \text{ kNm/m}$$

and the design moment of resistance of the inner leaf

$$=\frac{0.45\times1.67\times10^6}{3.5}=0.21 \text{ kNm/m}$$

Note that there is no need to consider bending about the plane of failure parallel to the bed joints, since the code method of design for two-way spanning panels automatically deals with this, ie, if the panel is satisfactory in the perpendicular to bed joint failure case, it will also be satisfactory in the parallel to bed joint failure case.

36.4.5 The design moments on each leaf may be calculated assuming that the applied horizontal force is shared between the two leaves in proportion to their design moments of resistance.
Therefore:
proportion of design wind load taken by the outer leaf

$$=\frac{0.75}{0.75+0.21}\times0.92=0.72 \text{ kN/m}^2$$

and the proportion taken by the inner leaf
$=0.92-0.72=0.20$ kN/m^2

36.4.2 *Design bending moments*

The design bending moment on a laterally loaded wall
$=\alpha \gamma_f W_k L^2$.
From code Table 9, case B, for the outer leaf the bending moment, α,
$=0.060$ for orthogonal ratio of 0.33 and aspect ratio, $\frac{h}{L}$, of 0.75,
for the inner leaf, the bending moment coefficient, α
$=0.054$ for orthogonal ratio of 0.56 and aspect ratio of 0.75.
Design bending moment on outer leaf
$=0.060 \times 0.72 \times 4.0^2=0.69$ kNm/m.
This is less than the moment of resistance of this leaf, and is therefore satisfactory.
Design bending moment on inner leaf
$=0.054 \times 0.20 \times 4.0^2=0.17$ kNm/m.
This is less than the moment of resistance of this leaf, and is therefore satisfactory.

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