BS 5628: The Structural Use of Masonry
Part 1: Unreinforced Masonry

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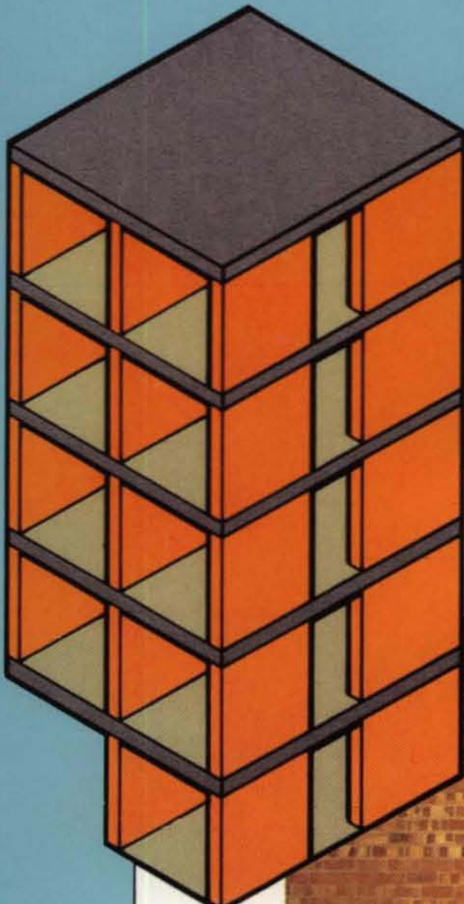
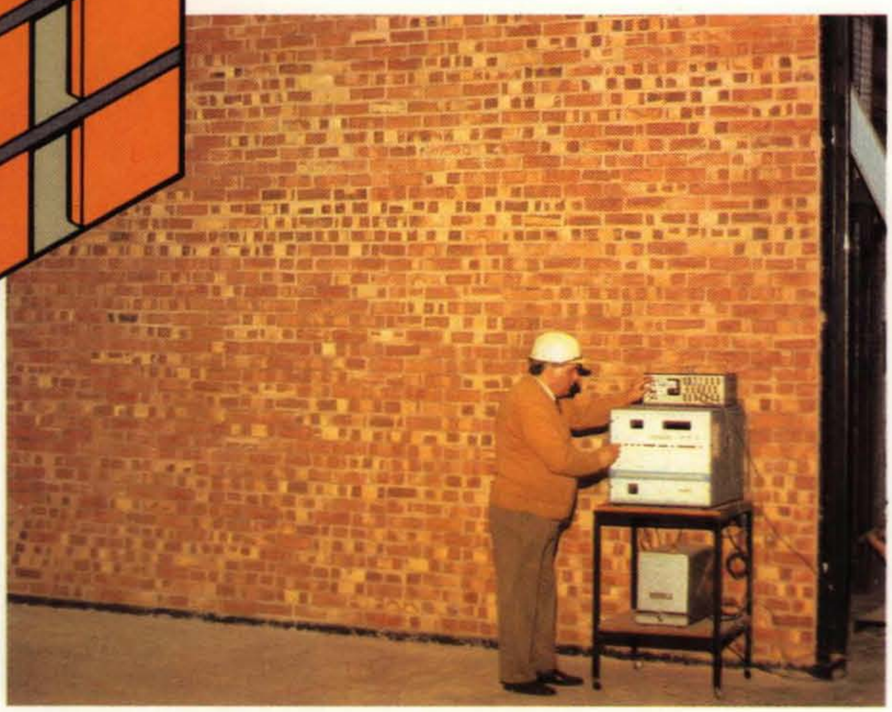
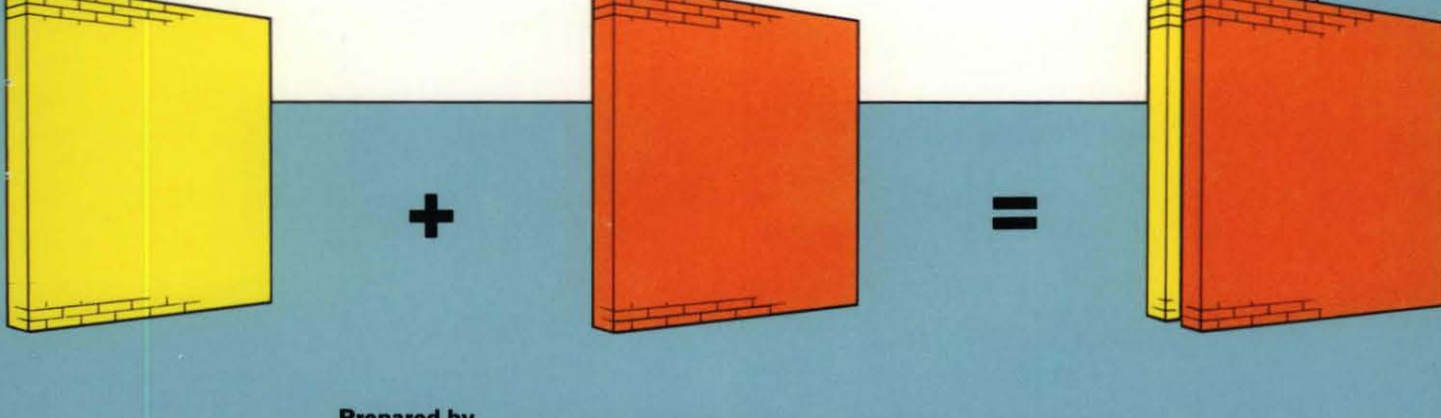
February 1986

The design of laterally loaded walls







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The design of laterally loaded walls

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FOREWORD

This publication is concerned with the subject of laterally loaded walls, with particular reference to uniform lateral pressures. It is based on visual presentations originally given during a series of seminars on BS 5628: The Structural Use of Masonry: Part 1 in late 1978 and subsequently.

Scope

The contents cover both the background to the Code provisions as well as the provisions themselves. In order to give the reader an understanding of the Code recommendations and the reasoning behind them, the subject is dealt with in its widest sense.

Particular attention is paid to Clause 36 of Section 4 of the Code which gives detailed design recommendations for laterally loaded walls.

Amendments 2747 (October 1978) and 3445 (September 1980) have been taken into account, as has the latest amendments 4800, March 1985.

Acknowledgements

The kind support and guidance of many people during the preparation of both the seminars and this publication is gratefully acknowledged. Particular thanks are due to B.A. Haseltine, current chairman of the Technical Drafting Committee, and to N.J. Tutt who checked the design examples.

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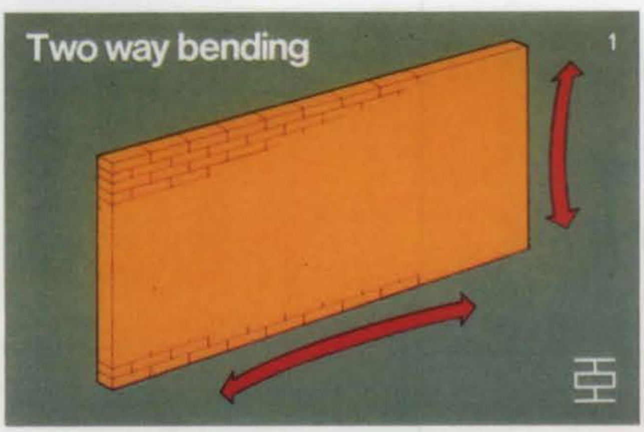
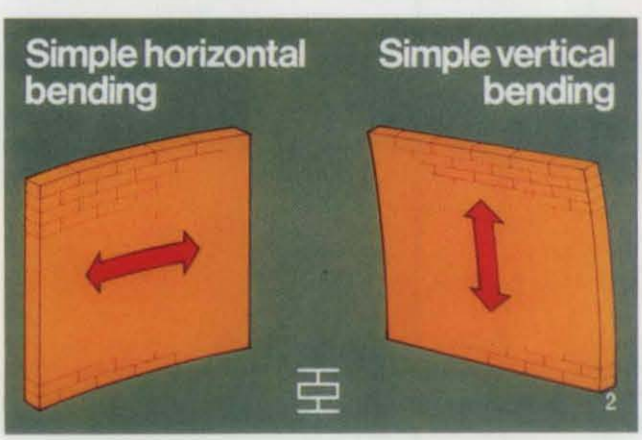
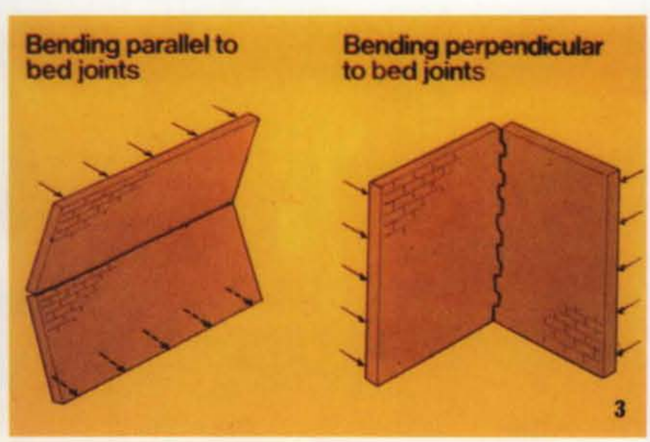
Introduction





The majority of masonry cladding panels tend to bend in both the horizontal and vertical directions simultaneously; they are essentially two-way bending plates (figure 1). It is more difficult to study the material properties in both directions simultaneously than if the panel is split into its horizontal and vertical modes and each of them studied separately. This facilitates any experimental investigation (figure 2).



Ideally, walls like those shown above should be built and tested in bending to failure. The failure mode will be a flexural crack that will occur along the bed joint, in the case of simple vertical bending (left) at or near the position of maximum moment. When the wall is subject to simple horizontal bending, the flexural failure crack will develop through the perpendicular joints (perpends) and the bricks, as shown on the right.

While full scale model tests are perhaps ideal, they are also expensive. It is more cost effective to build and test small walls, known as wallettes.

Two wallettes and a test rig can be seen above. The wallette, built to a particular format, in the rig in the background is being tested in simple vertical bending. The wallette in the foreground, built to a different format, is to be tested in simple horizontal bending and will be tested in a different rig.

Weak and strong direction of bending

The wallette in the test rig in figure 4 will fail, at or near mid-height, when the interface between the bricks and the mortar bed is broken. Since the brickwork has relatively weak tensile properties in both direct and bending tension, it is reasonable to predict that this direction of bending (simple vertical) will also be relatively weak. On the other hand, the wallette in the foreground, when loaded to failure in the other rig, will fail when a vertical crack develops across two bricks and two perpends. Since a brick is much stronger in flexure than a mortar/brick interface, it is reasonable to predict that this direction of bending will be relatively strong.

Experimental evidence confirms that simple vertical bending is the weak direction, and that simple horizontal bending is the strong direction. For ease of terminology, the direction of horizontal bending and the direction of vertical bending will now be referred to as the strong and the weak directions of bending respectively.

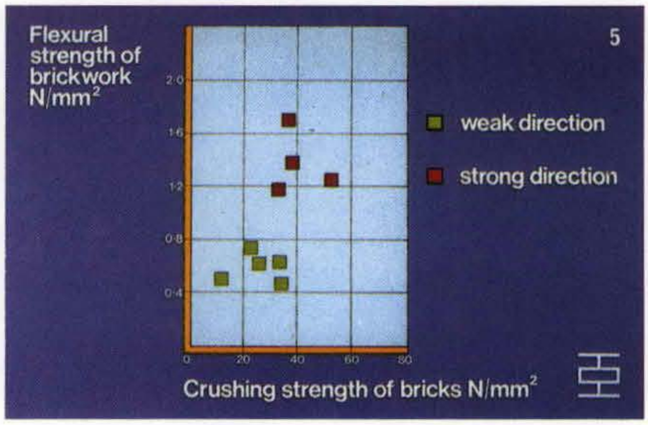
Wallette test programme

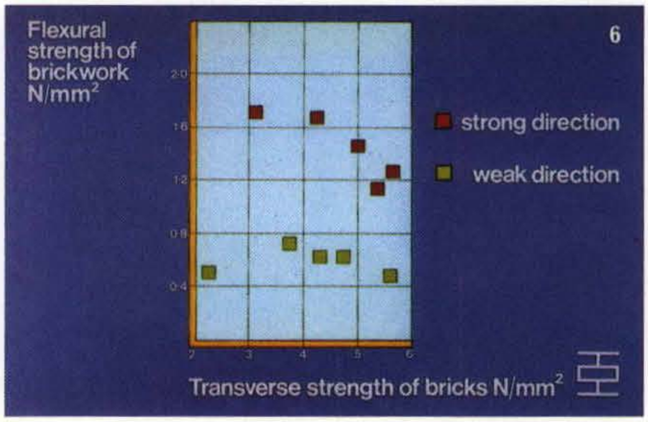
The wallette testing programme was carried out by the British Ceramic Research Association[1]. A wide variety of bricks, commonly available in Britain, was tested in the four British Standard mortar designations. The objective was to ascertain the flexural strength, in both the weak and strong directions of bending, for the range of modern brickwork designed and constructed in this country.

Clay bricks

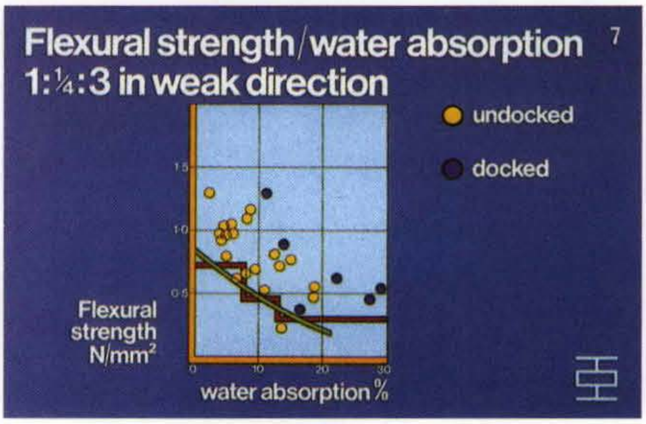
For each clay brick/mortar combination, a set of wallettes was built in both the horizontal and vertical formats, each consisting of five or ten wallettes. This enabled the mean and standard deviation of each set of results to be found. With these values for the mean failure stress and the standard deviation, the characteristic flexural stress could be calculated for the strong and the weak direction of bending. But to what parameters could these results be related?

Perhaps the best known property of bricks is their compressive strength. Does the flexural strength of brickwork relate to the compressive strength of the bricks? The relationship is shown in figure 5. From the limited results available, it was not felt to be a relevant approach.





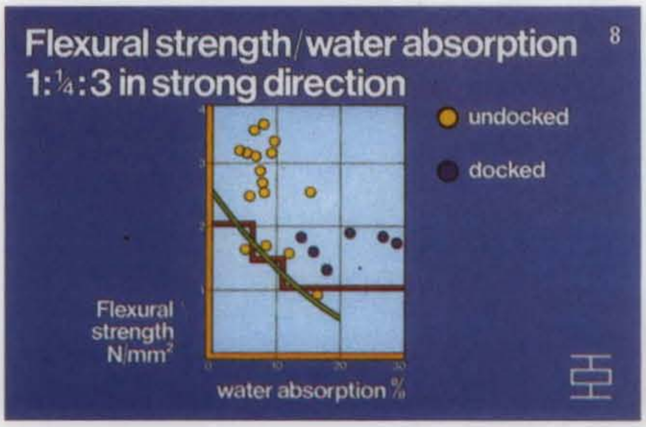
It is reasonable to predict that some relationship would exist between the strength of the brick in flexure and the flexural strength of the brickwork – particularly in the strong direction, if not in the weak. Figure 6 shows that there is some correlation between these parameters.



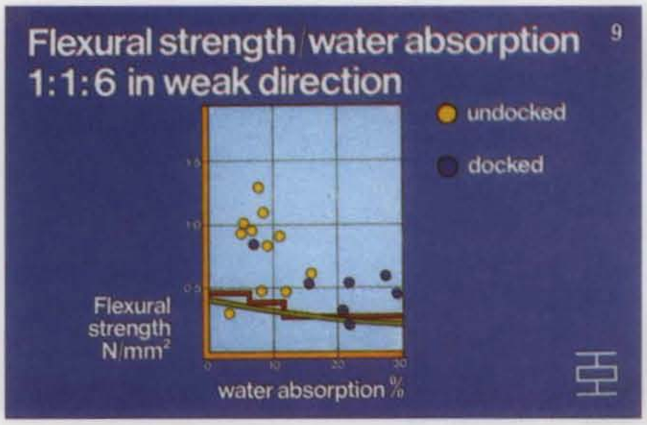
After examination of all the possible relationships, it was decided that the relationship between the flexural strength of the brickwork and the water absorption of the bricks[2] from which it was constructed would be used [1]. The graph above relates the flexural strength of the brickwork to the water absorption of the brick for wallettes tested in the weak direction and built with designation (i) mortar. Each point on the graph represents the mean of five or ten wallette tests with a particular brick. The number of dots indicates the wide range of bricks that were tested in order to represent the wide variety of clay bricks available in Britain to-day. The wide scatter is not exceptional; it is to be expected when testing masonry. It reflects the fact that brickwork has a wide statistical 'quality' curve.

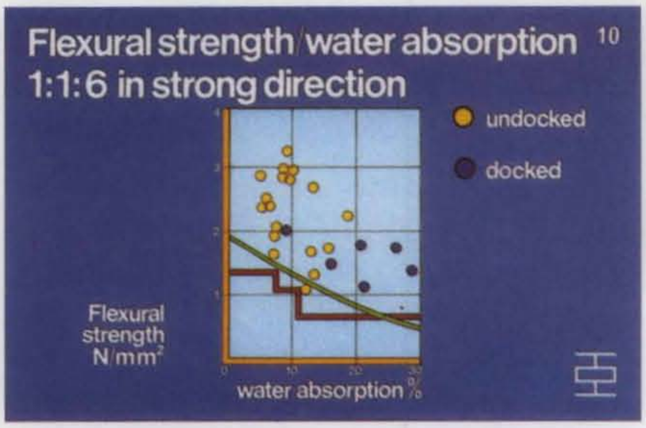
To comply with limit state philosophy, it was necessary to establish the 95% confidence limit from all the test results. A statistically based approach was used to establish the position of the 95% confidence limit from the mean and standard deviations associated with each point. This limit is shown in figure 7 as the curved line, under which lie 5% of the results. It can be seen that the flexural strength tends to decrease with an increase in the water absorption of the bricks. Because complex curves make design less straightforward, it was decided to simplify the curve with an 'approximation'. This is shown as a straight dotted line with two steps at values of 7% and 12% water absorption. This convenient approach of having three values of flexural strength for three ranges of water absorption, facilitates the material properties of brickwork being described in tabular form.

It is from this research background that the characteristic flexural stresses given in Table 3 of the Code were derived. This is reproduced as figure 13 on page 5. From both the graph and the table, these values are 0.7 N/mm^2 (W.A. less than 7%), 0.5 N/mm^2 (W.A. greater than 7% and less than 12%), and 0.4 N/mm^2 (W.A. greater than 12%) when considering the weak direction of bending in mortar designation (i).



Looking at another similar graph, figure 8, this time in the strong direction – but still with mortar designation (i) – the background to another section of Table 3 of the Code (figure 13) can be seen. The approach adopted is identical to that described above and the values on this graph correlate with those in Table 3.





The relationship between the characteristic flexural stresses of brickwork – in the weak and strong directions – and the water absorption of the clay bricks from which it was constructed was established for all the mortar designations. Figures 9 & 10 show the graphs for a designation (iii) (1:1:6) mortar. As before, figure 13 contains the values derived from this work.

Calcium silicate bricks

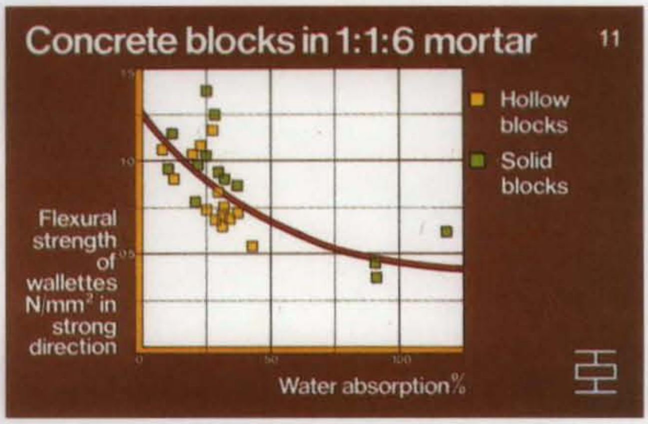
The approach for calcium silicate bricks was essentially identical to that used for clay bricks. Wallette sets were built, in both formats, using a range of calcium silicate bricks representative of modern production in Britain. They were constructed using four different mortar designations. The results of the large testing programme indicated that only one set of values for the characteristic flexural stress was needed to cover all calcium silicate brickwork. These values, 0.3, 0.2, 0.9 and 0.6 N/mm^2 for the respective mortar designations, are given in figure 13. Calcium silicate bricks available in Britain do not, of course, have a water absorption value associated with their Standard[3].

Concrete bricks

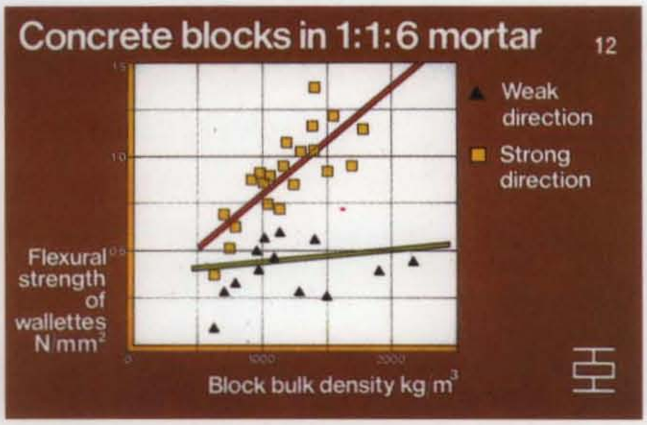
Concrete bricks were tested in an identical manner to the other brick wallette tests. The results, see figure 13, indicate similar values to those associated with calcium silicate brickwork.

Concrete blocks

Concrete blocks are covered in Table 3 (figure 13) in a somewhat different manner to clay, calcium silicate and concrete bricks. While the characteristic flexural stress was ascertained from wallette tests, a different wallette format was necessary to account for the difference in unit size between bricks and blocks. The wallette formats for blocks are given in Appendix A3 of the Code.



As with calcium silicate and concrete bricks, concrete blocks do not use the parameter of water absorption in their Standard[4]. While the wallette test programme indicated that there was some relationship between water absorption and flexural strength (figure 11), water absorption was not considered to be the most relevant indicator of the flexural strength of blockwork.



The relationship between block bulk density and the flexural strength was also examined to see if bulk density might be a useful indicator. While certain trends are apparent in figure 12, they were not felt to be as relevant as the relationship between flexural strength and the compressive strength of the block itself. For this reason, the flexural strength of blockwork in figure 13 is given in terms of the crushing strength of the block.

Considering the strong direction of bending, it must be remembered that the failure of the wallette involves the failure of both the perpend joints and the blocks. It is reasonable to expect an increase in flexural stress with an increase in block strength. This is because the stronger the block in compression, the denser it will be. The denser the block, the greater the flexural strength of the block itself. And the greater the flexural strength of the block, the greater will be the influence on the strength of the wallettes when tested in the strong direction. This trend was found to be significant in the strong direction. It is reflected in figure 13 which bases flexural strength on the compressive strength of the block. The characteristic flexural stress values range between 0.9 to 0.4 N/mm^2, and 0.7 to 0.4 N/mm^2 depending on the mortar designation for 100 mm blockwork.

In the weak direction, failure occurs by breaking the tensile bond at the interface between the blocks and the mortar bed. The experimental evidence suggests that there is no significant difference between blockwork constructed with weak or strong blocks. Consequently, for 100 mm block work there are only two values of 0.25 N/mm^2 and 0.2 N/mm^2 which are dependent on the mortar designation but independent of the strength of the block itself.

Since the Code was first published, further tests have indicated that the flexural strength of thicker wallettes is lower than that predicted by the wallette results for 100 mm blockwork. The Standard has therefore been amended to take account of this.

For intermediate values of wall thickness, between 100 and 250 mm, the value of characteristic flexural stress can be obtained by interpolation.

Characteristic flexural strength of masonry f_{kx} N/mm^2

Mortar designation	Plane of failure parallel to bed joints (i)	(ii)(iii)	(iv)	Plane of failure perpendicular to bed joints (i)	(ii)(iii)	(iv)
Clay bricks having a water absorption less than 7%	0·7	0·5	0·4	2·0	1·5	1·2
between 7% and 12%	0·5	0·4	0·35	1·5	1·1	1·0
over 12%	0·4	0·3	0·25	1·1	0·9	0·8
Calcium silicate bricks	0·3		0·2	0·9		0·6
Concrete bricks	0·3		0·2	0·9		0·6

Concrete blocks of compressive strength in N/mm^2	Parallel (i)(ii)(iii) t=100	t=250	Parallel (iv) t=100	t=250	Perpendicular (i)(ii)(iii) t=100	t=250	Perpendicular (iv) t=100	t=250
2·8	0·25	0·15	0·2	0·1	0·4	0·25	0·4	0·2
3·5					0·45	0·25	0·4	0·2
7·0					0·60	0·35	0·5	0·3
10·5	0·25		0·2		0·75		0·6	
14·0 and over					0·9†		0·7†	

* For values of thickness between 100mm & 250mm interpolation is permitted
† When used with flexural strength in parallel direction, assume the orthogonal ratio μ=0·3

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Orthogonal ratio

The orthogonal ratio is the ratio of the strength in the weak direction to the strength in the strong direction. For clay, calcium silicate and concrete bricks, the ratio is approximately 1:3 or 0.33 (see figure 13). For design purposes, a value of 0.35 can be universally adopted for all these bricks.

Unfortunately, with concrete blocks, the ratio varies since the strong direction stresses vary whilst the weak direction stresses remain constant for different strength blocks. There is, therefore, no unique value for the orthogonal ratio of blockwork; it is necessary to establish a value for each blockwork strength.

Statistical quirks

There are certain implications inherent in a statistical approach. The graph shown in figure 10, whilst being statistically correct, does nonetheless misinterpret certain facts. Both the curved line and the stepped approximation indicate that the strongest brickwork results when bricks with a water absorption of less than 7% are used. A closer examination of the graph, however, reveals that brickwork constructed with bricks of between 7% and 12% water absorption can exhibit higher values of flexural strength than can be achieved with many of the bricks with less than 7% water absorption. Indeed, the largest value of flexural strength is achieved with a brick in the water absorption range 7%–12%.

It is because of such statistical quirks, that the Code permits wallette tests to be carried out on particular bricks in accordance with the testing procedure set out in Appendix A3. In this way, a manufacturer's claim of higher characteristic flexural strengths than those in the Standard can be objectively assessed by the designer if, at his discretion, he decides to ask for wallette results to BS 5628: Part 1 from the material producer.

Docking or wetting of bricks

The graphs shown in figures 7, 8, 9 & 10 make reference to 'docked' and 'undocked' bricks. This is due to the need to wet ('dock') certain types of brick if their water absorption would result in water being removed from the mortar at too great a rate. Research has indicated that this high speed removal of water from the mortar can adversely affect both the compressive strength of the brickwork and the bond at the brick/mortar interface.

The rate at which water is removed from mortar is called the Initial Suction Rate of the brick (ISR). The ISR of a brick can be adjusted by *partially* soaking it. This can be done either by immersing it in water for a short time – typically, a half to three minutes. It can also be partly achieved by sprinkling the stack with water from a hose. Although this is often done on some sites, it is *not* the recommended method. When writing the specification, it is normal good practice to suggest that 'before orders for bricks are placed, the contractor shall satisfy the engineer either that the suction rate of the brick ... does not exceed 1.5 kg/m^2/min or that he is able to adjust it so as not to exceed this value'[5].

'Docking' or wetting of bricks does *not* mean saturating them. The general rule '*never* allow bricks or brickwork to become saturated' should always be obeyed.

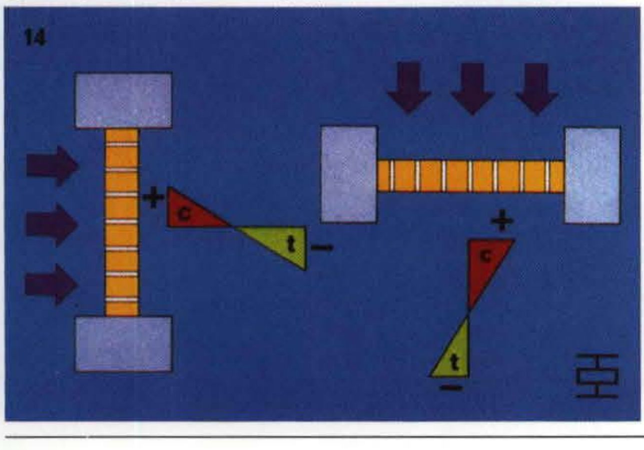
Summary

The foregoing material constitutes the background to the section of the Code which deals with material properties. In particular, it explains the background to Table 3 (figure 13), and provides more detailed information on the large testing programme on which it was based. Not all the graphs have been shown since they would give no more information than is contained in Table 3.

DESIGN METHOD 2

Introduction

Turning from the consideration of material properties to the design of panels using these characteristic flexural stresses, two types of panel are particularly important.



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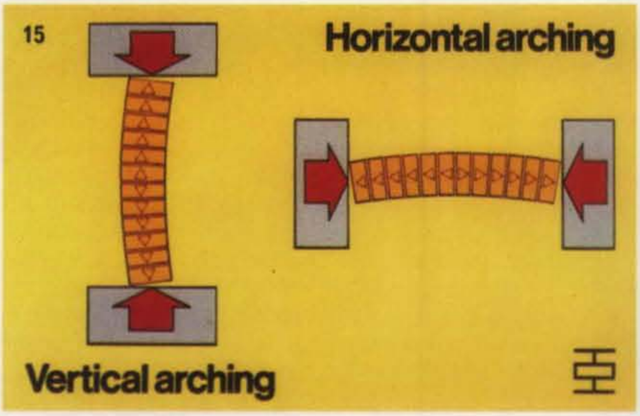
Cladding panels

The first type is the panel which fails when the tensile stress in the extreme fibres equals the ultimate stress. It could be a vertically spanning wall as shown on the left of figure 14, or a horizontally spanning panel as shown on the right. This type of wall is very common in Britain, and the vast majority of brickwork panels used to clad framed buildings fall into this category.

Panels which arch

The other type of panel, which behaves differently from a cladding panel, is the panel that 'arches'. Such a wall (figure 15), generates compressive forces within the plane of the wall as it deflects. These compressive stresses, generated by the deflection, are superimposed on the tensile stresses as they develop and partly or wholly cancel them out. Consequently, this type of panel is much stronger than a cladding panel.

A particular type of wall panel in which in-plane forces are present is a loadbearing wall in a structural masonry building. In this case, the in-plane compressive stress present is the stress in

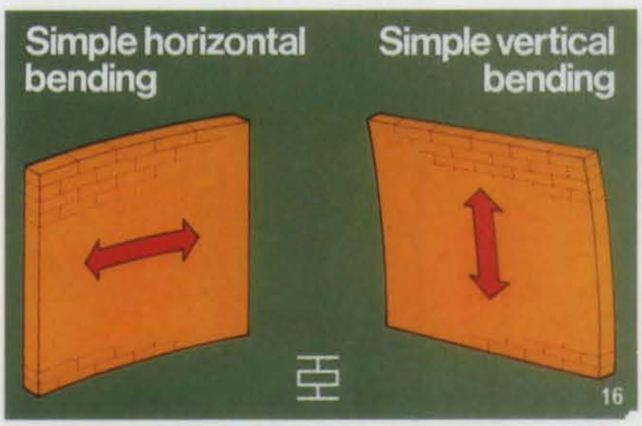


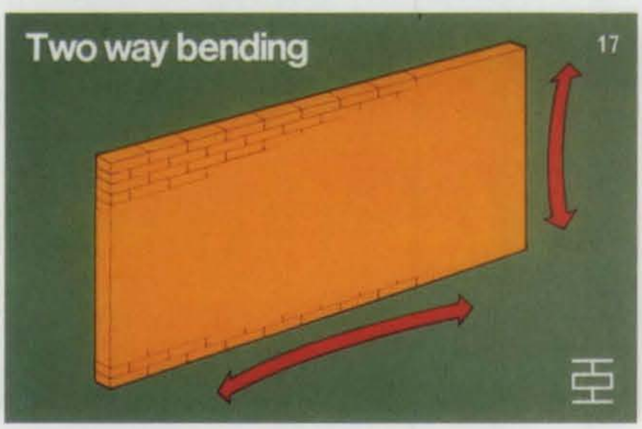
the wall derived from the load it is supporting.

It is important to distinguish which type of wall is being designed, and whether it is a wall in which compressive forces can develop. This will now be considered in greater depth.

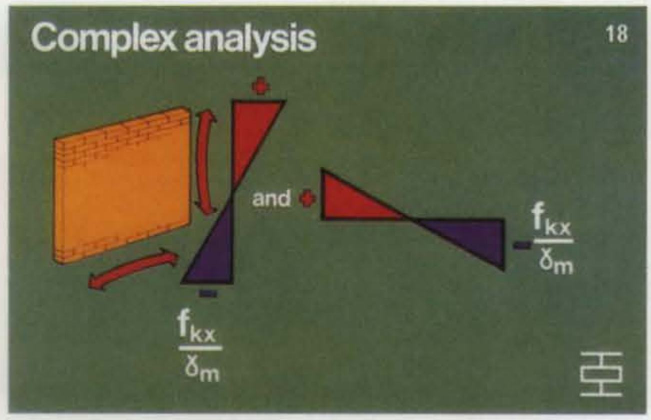
Cladding panels

The first type of wall to consider is the cladding panel that fails when the tensile stress developed at the extreme fibres in bending reaches the ultimate flexural stress for the material. Having already established the material properties that can adequately predict the simple vertical and horizontal bending strengths (figure 16), it is now necessary to combine them in two-way bending (figure 17). This essentially relates the two known material properties described in Table 3 of the Code to the behaviour of three and four-sided plates.



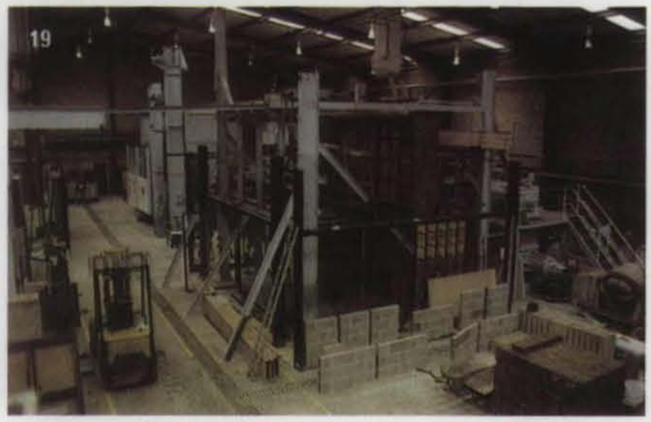


It is a complex analysis and involves the summation of the two stresses generated from two-directional bending (figure 18).



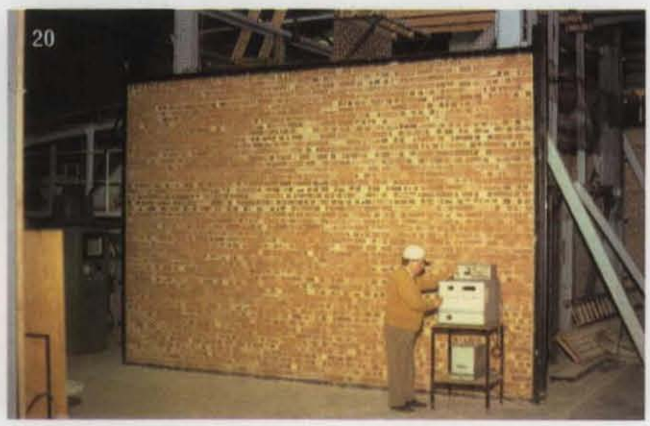
Wall tests

To investigate this, a series of full scale wall tests were carried out at the British Ceramic Research Association[1] (figure 19). Wallettes awaiting testing can also be seen in the foreground of the picture.



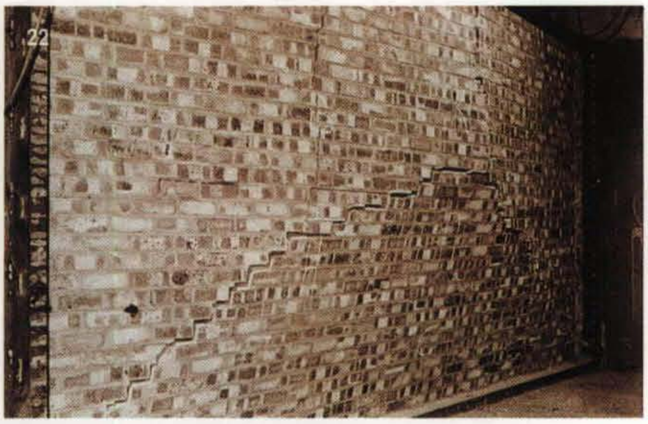
The objective was to find the relationship between the strength in two-way bending to the two known strengths in the simple vertical and horizontal directions.

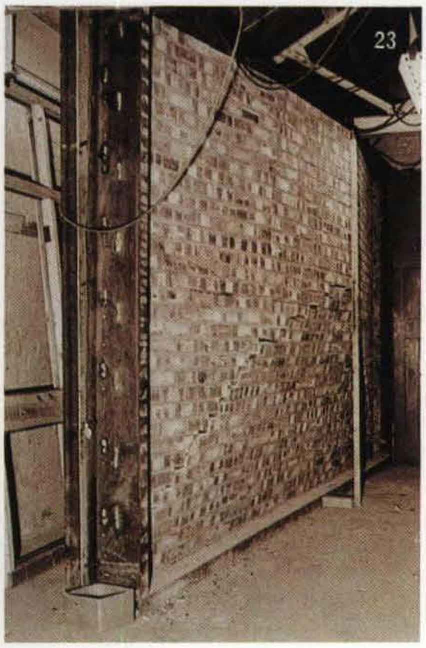
Full scale wall panels were built in test rigs (figure 20) and an air bag was placed between the wall and a reaction frame behind it (figure 21). The reaction frame was tied back to the test rig. Panels of different shapes were then tested using various brick/mortar combinations. The majority of the panels built were standard storey height (figure 21) but some taller and shorter panels were also tested, such as the 1½ storey height panel in figure 20. The length of the panels was also to be varied – note the two shorter panels awaiting testing in the foreground of figure 21.



Edge supports

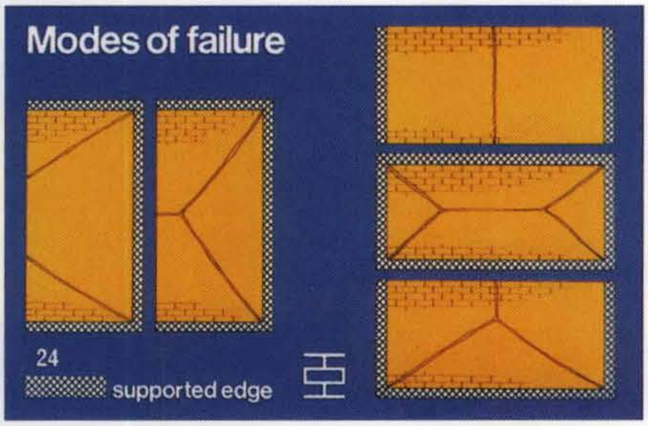
Actual failure, when the panel cracks, does not necessarily result in total collapse. The wall in figure 22 actually cracked at a much lower deflection. In order to demonstrate the failure pattern more clearly, the air bag was inflated still further until the





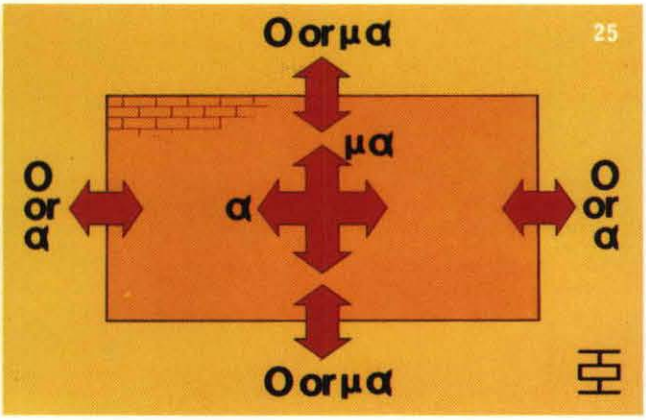
wall was grossly deflected (figure 23). This also serves to illustrate the residual strength of failed panels; although the wall has structurally failed and is grossly deflected, it is stable. Visible in figure 23 are the holes that were cut in the channels forming the two vertical edges of the test rig. Ties were inserted into these holes and were used to connect the wall to the frame, as is done in practice – ie, the panels were supported in the test frame in a similar way to panels in a real building. It was of course recognised that this edge support system was more likely to approximate to simple supports rather than to fully fixed edge conditions.

Failure modes



The failure modes of the panels (figure 24) gradually emerged as the test programme progressed. They were found to be fairly similar to those associated with concrete slabs although, of course, brickwork is a brittle material (unless reinforced) while reinforced concrete exhibits plastic yielding.

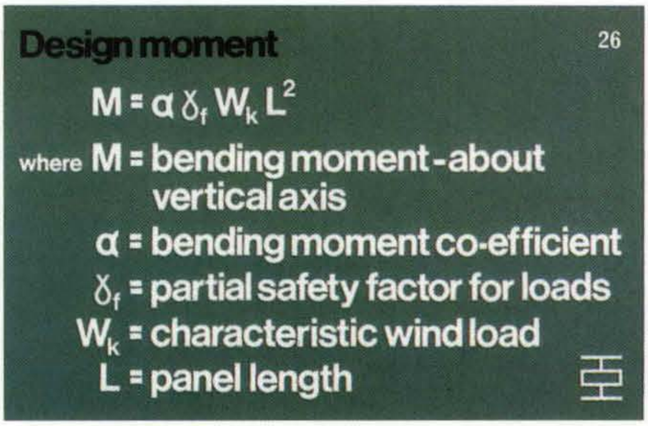
It was recognised that, if account could be taken of the two different bending moments at the centre of the panel in both directions, and also for various possible edge conditions (figure 25), a useful design technique would result. The three possible edge conditions would be:



1. A free or unsupported edge.
2. A supported but pinned edge where no moment would develop.
3. A supported fully fixed edge where a moment equal to the panel mid-span moment would develop.

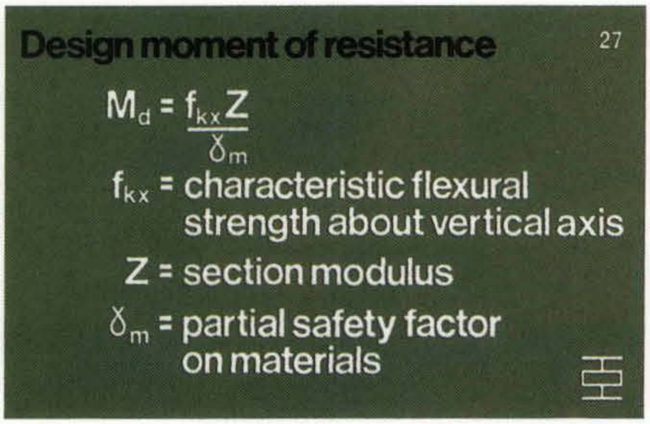
Working on this basis, and taking into account the wall test results, the following design procedure was suggested[6]:

Design moment



In the design of any bending member, the applied moment is normally WL^2 divided by a factor, for example 8 for simply supported one way bending. The design moment for panels is based on the same equation but the divisor is expressed as an α coefficient (eg, 0.125 for the case above). For limit state design γ_f, the partial safety factor for loads, is necessary to convert the applied moment into an (applied) design moment. Note that the formula is expressed in terms of L, the length, and *not* h, the height.

Design moment of resistance

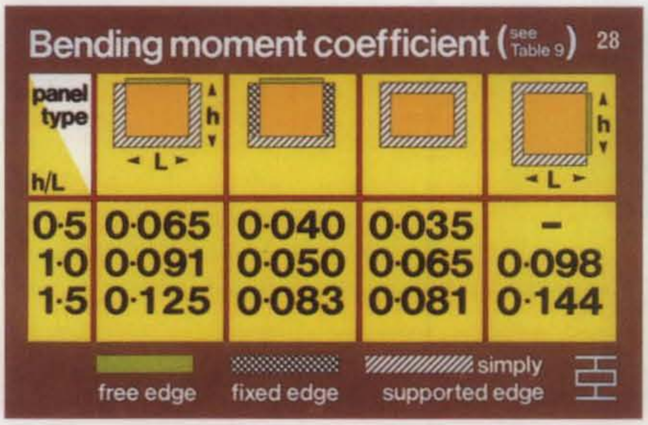


The formula to calculate the moment of resistance is the normal f x Z term. Because the applied moment uses L (ie, it is associated with horizontal spanning), the moment of resistance must use f_{kx} in the strong direction for compatibility. To be a design moment of resistance, the f x Z term must be divided by

γ_m, the partial safety factor for materials.

By equating the design moment of resistance to the design moment, it is now possible to objectively design laterally loaded panels. But it is first necessary to look in detail at certain of the terms used.

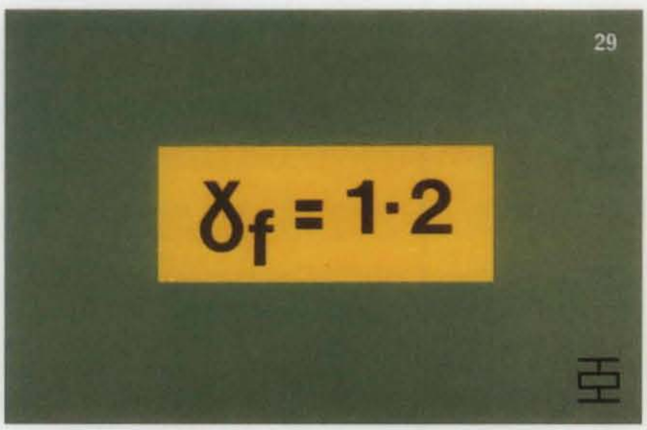
Bending moment coefficient, α



h/L				
0·5	0·065	0·040	0·035	–
1·0	0·091	0·050	0·065	0·098
1·5	0·125	0·083	0·081	0·144

Bending moment coefficients are set out in Table 9 of the Code. They are essentially the coefficients generated from 'yield line' theory[7,8] which, although not felt to be strictly applicable to brittle materials such as masonry, nevertheless appear to give reasonable correlation with experimental results – at least for solid panels without windows or doors[6]. The α coefficient depends on the aspect ratio of the panel (h/L), on the orthogonal ratio μ, and the nature of the panel's edge support – whether three or four-side supported, and whether 'simply supported' or 'fully fixed'. (Note that a set of values for α has been given in each of the tables for μ = 0.35. This value for μ has been purposely included for all types of brickwork.)

Partial safety factor for load (wind), γ_f



Normally, γ_f for general design is taken as 1.4. However, a special case is made of cladding panels which do not affect the stability of the main structure – normally a frame. In this case, γ_f can be reduced to 1.2. It must be clearly understood that this is an exception, based not on design philosophy but on the need to acknowledge that without reducing γ_f to 1.2 many panels currently known to work well in practice could not be justified in the design process. Of course, should the brickwork panel form part of the *structure*, and should the removal of the panel endanger the stability of the remaining structure, γ_f must be maintained at 1.4.

Checking the design

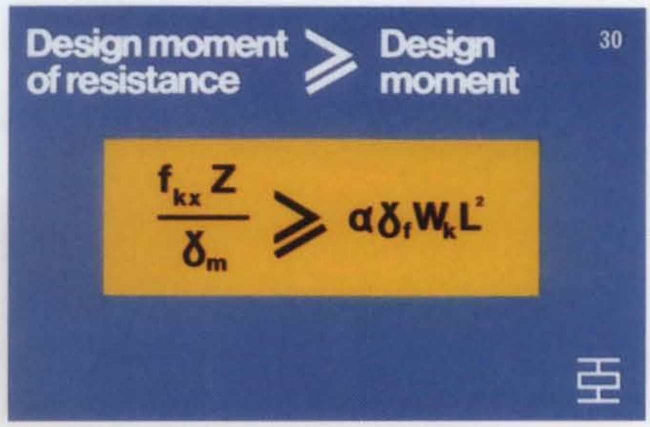
The design moment of resistance must be equal to or greater than the design moment and, by re-arranging the formulae:

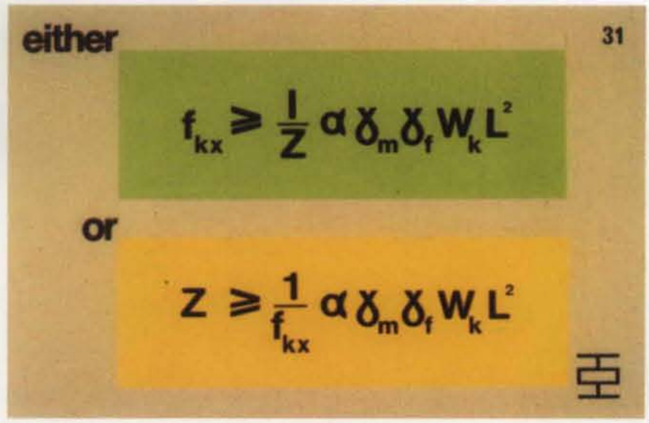
either,

Z is assumed and the f_{kx} of the material is checked to see if it is adequate,

or,

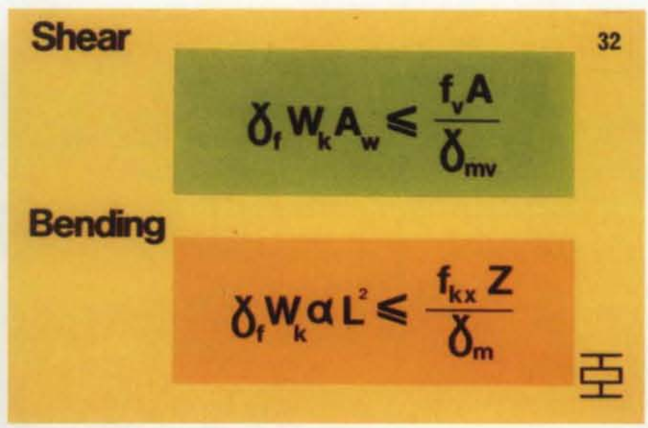
the f_{kx} of the material to be used is fed into the design and the Z value of the section to be used is checked.





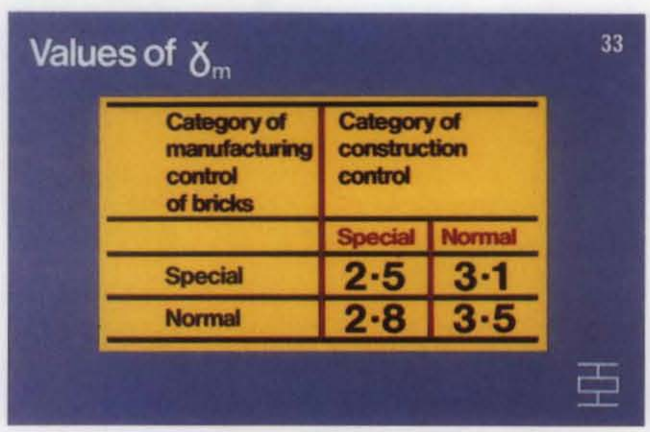
These are normally the two main ways in which the design is carried out.

Shear



For the majority of panels met in practice, failure in bending will be the design criterion. It is nevertheless necessary to check the shear. The above formula gives this readily.

Partial safety factor materials, γ_m



Category of manufacturing control of bricks	Category of construction control	
	Special	Normal
Special	2·5	3·1
Normal	2·8	3·5

The value of γ_m can vary between four values depending on the category of construction control (workmanship and supervision) and the category of manufacturing control (the likelihood of weak units being included in the consignment of structural units). This approach essentially recognises that there are bonuses for the designer if he can be more certain about the structural units, the mortar and the way they are put together on site. These design bonuses can be exploited if:

1. There is a smaller probability of the structural units (bricks/blocks) falling below their specified strength.

2. There is a smaller, probability of the mortar being below the strength specified.

3. There is a smaller probability that poor workmanship will be used in putting the bricks and mortar together on site.

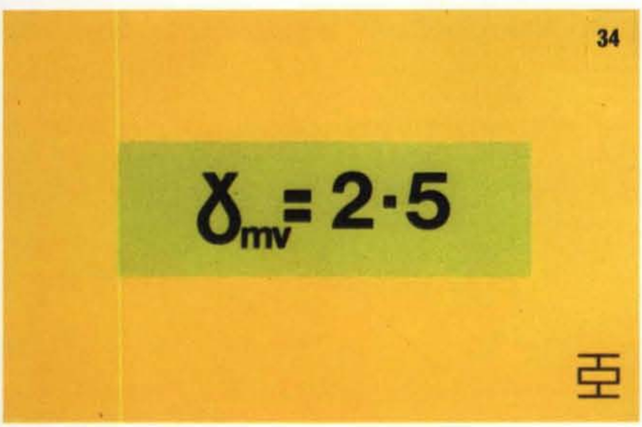
Condition 1 is covered by the manufacturing process, while 2 and 3 are both factors influenced by site operations and supervision.

As is the case with compression and shear loaded walls, it is possible to specify bricks to an *acceptance* limit to BS 3921[2] where not more than 5% of the bricks will have a crushing strength below the acceptance limit specified. This is the requirement for special category of manufacturing control, and the majority of BDA member companies will readily be able to meet this form of production quality control.

To achieve special category of construction control, mortar testing should be carried out to satisfy condition 2, above, *and* the work on site should be supervised by a suitably qualified person to satisfy condition 3.

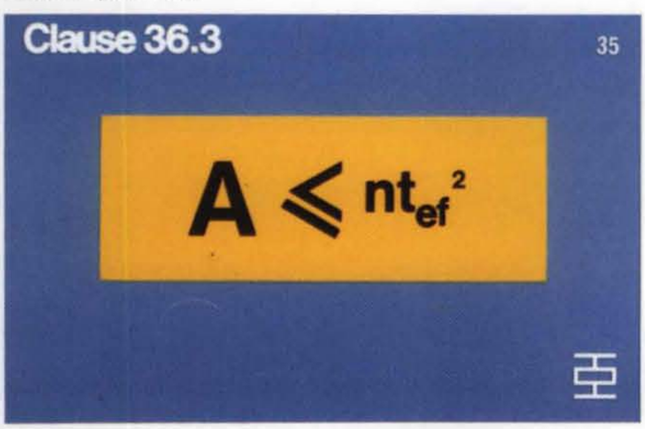
In most situations, the designer will normally base his design on $\gamma_m = 3.5$ or 3.1 since, at the design stage, the likely contractor and the level of site supervision will not necessarily be known. However, lower values of γ_m are available, should they be required.

γ_{mv} for shear



γ_{mv} for shear is taken as 2.5

Limits to panel size



Two limitations to panel sizes are given in the Code:

1. An overall limit on the area of the panel. Detailed restrictions are given, depending on the nature of the panel supports.

2. For three and four-sided panels, an overall limit on height or length which must not exceed $50t_{ef}$ (the effective thickness of the wall). Since t_{ef} for a normal cavity wall is approximately 137 mm, the practical limit for normal cavity walls spanning in both directions is about 6.5 m in length and height. (t_{ef}, the effective thickness of a cavity wall is normally $\frac{2}{3}(t_1 + t_2)$ where t_1 and t_2 are the actual thickness of the two leaves.)

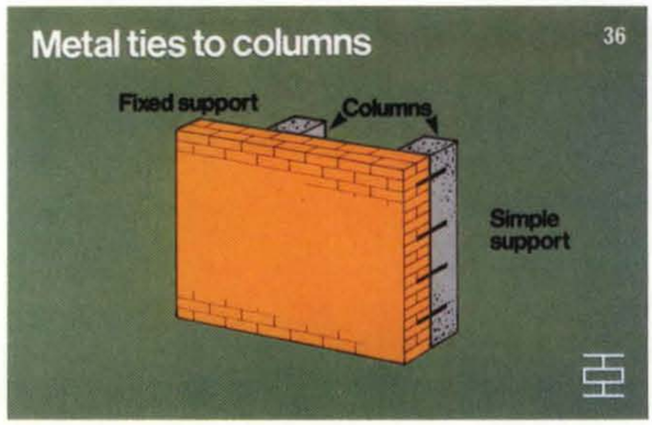
It is possible to build panels which exceed either or both 1 and 2 above, but they should be *effectively sub-divided* so that, while they may not appear to be two separate panels, they flex in bending as though they are. Normally, this sub-division is achieved using intermediate supports. In this way, conditions 1 and 2 can be effectively met.

Edge restraint

As with any structural member, the question of whether the ends are fully fixed or pinned is not necessarily easy to determine. The criterion used in the Code, however, is not difficult to grasp. If the edge of a panel is 100% fixed, such that, if loaded, the panel edge will fail in flexure before rotating, the panel is assumed to be 'fully fixed'. Anything less than this and the panel edge is assumed to be 'simply supported', even if in reality it may be partially fixed.

Looking at detailed cases:

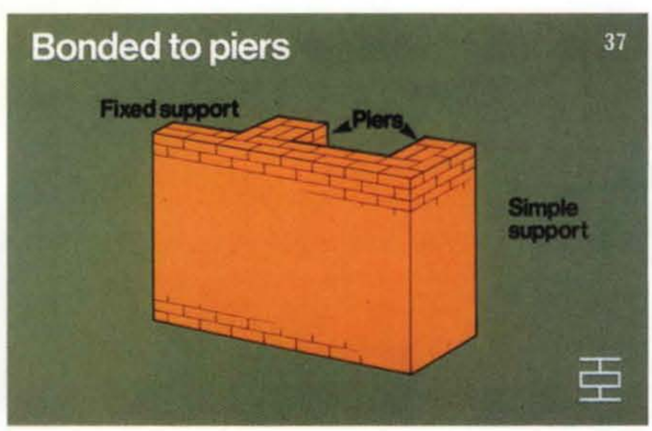
Metal ties to columns



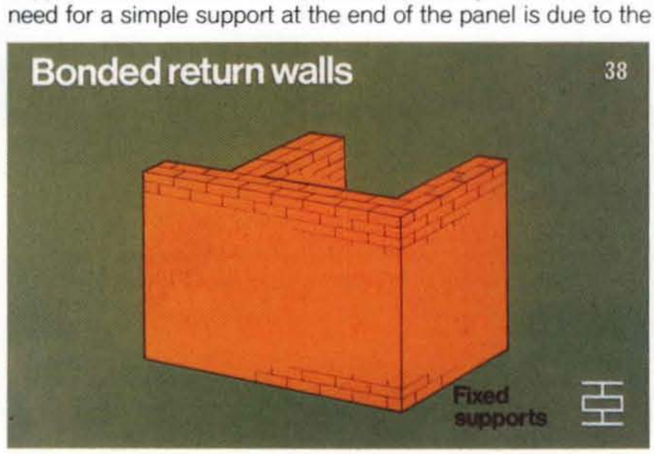
The non-continuous edge, suitably tied to a column, does not give 100% fixity against rotation. It is therefore assumed to be simply supported.

The fully continuous edge, achieved by taking the wall past the column, is 100% fixed. It will break along this vertical edge before it will rotate.

Bonded to piers



The above argument will also apply to piers. While the fixed support, where the wall is continuous, is readily understood, the need for a simple support at the end of the panel is due to the

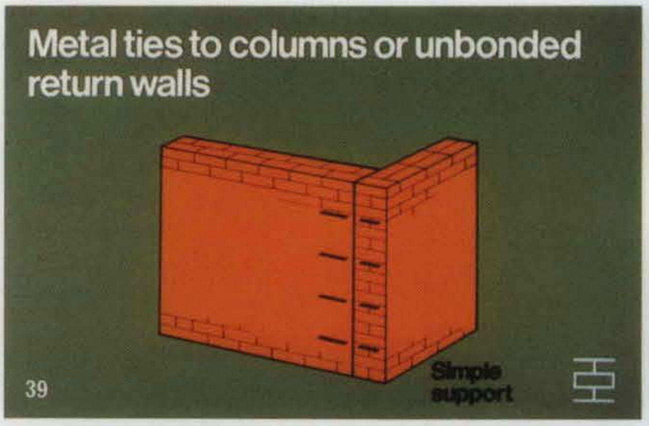


ability of the pier to rotate in torsion. If torsion restraint were present, the end could be considered fully fixed.

Bonded return walls

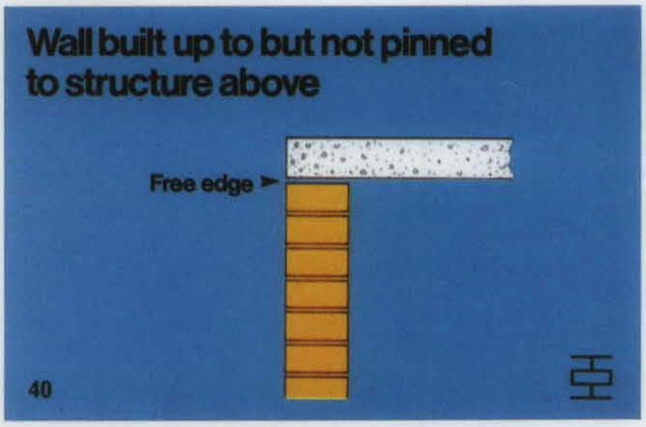
This full fixity is assumed if the pier becomes a *bonded* long return wall. Now both edges are fully fixed, and the wall must crack at the vertical supports before rotation can occur (figure 38).

Unbonded return wall using metal ties



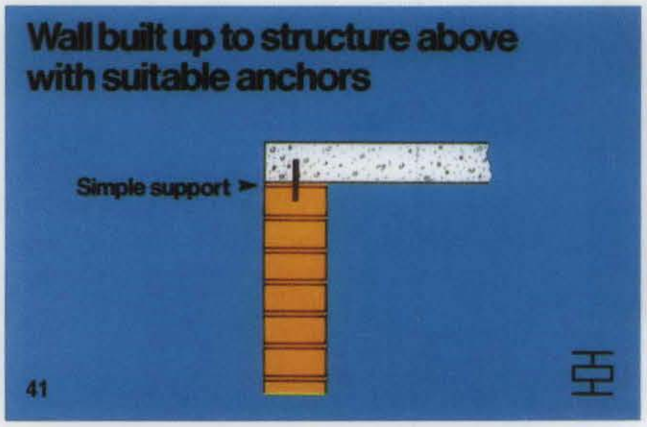
If the wall is not bonded but merely tied to the long return, some rotation will be possible. This vertical edge should be considered to be a simple support.

Free top edge



Irrespective of what happens at the vertical edges, the horizontal edges can also be considered as either fully fixed or simply supported. In the above example, the top edge is actually free – no support has been provided. This will be designed as a three-sided panel.

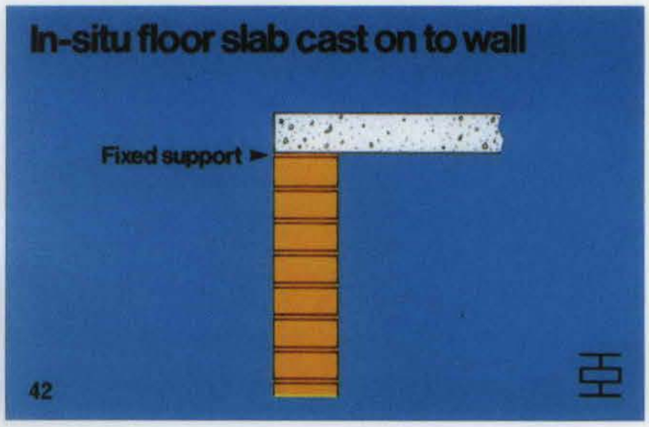
Wall built up to the structure with suitable anchorages



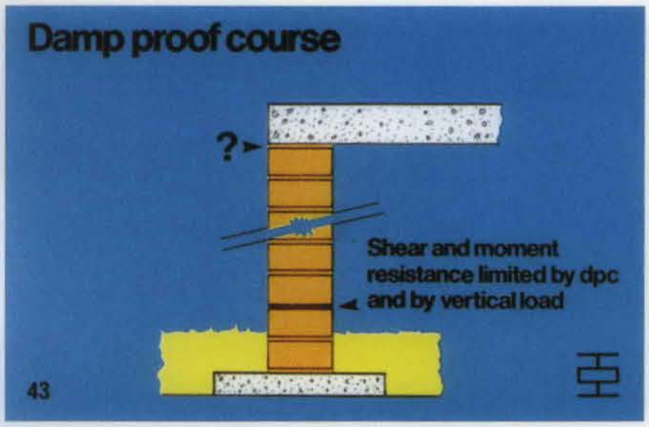
It is more usual to provide some form of support to the top edge. In the above example, the wall has been effectively pinned to the floor above. However, the top edge will still be able to rotate under lateral load and, therefore, it must be assumed to be simply supported.

In-situ floor slab cast on to wall

When an in-situ floor slab is cast onto a wall, the Code suggests that it can be considered fully fixed.



Effect of damp proof courses



Irrespective of the details at the top of the wall, it must be remembered that most panels have their shear and moment resistance limited at their bases by the need to introduce dpc materials. As a result, it is normal to expect the base of most panels to be simply supported.

Experimental validation

Whilst the above approximations, which give assumed edge conditions, are useful to the busy design engineer in practice, they are none the less over-simplifications.

In most of the cases considered where simple supports have been given, a degree of partial restraint is actually present. In many of the examples, 30%, 50% or 70% fixity may be present. Besides the obvious advantage of keeping design simple, this approach agrees with the published experimental evidence[6].

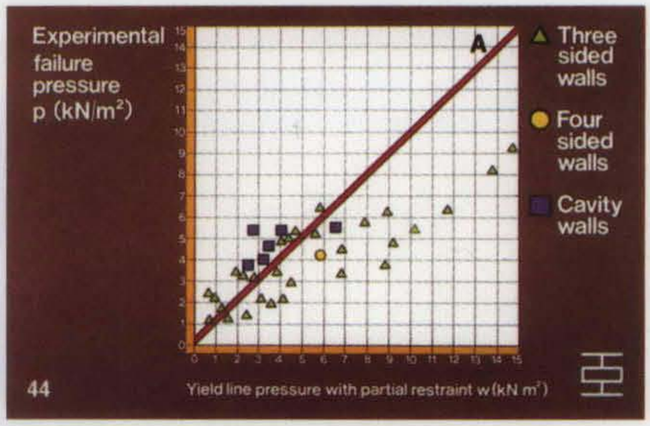
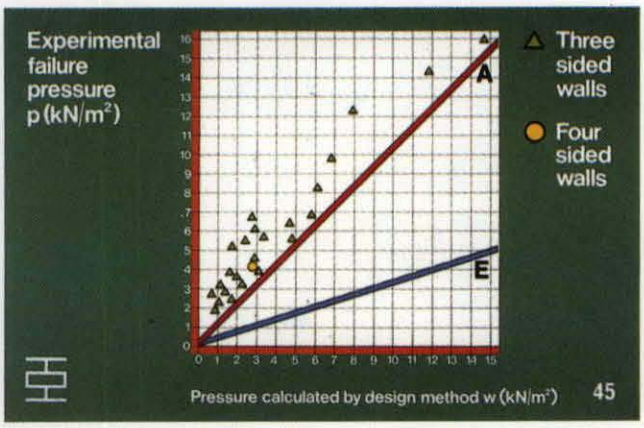


Figure 44 compares the predicted failure load, based on the design method outlined previously, but using an estimation of the degree of edge restraint, with the experimental results of walls tested in the laboratory. Whilst the correlation is reasonable for failure loads of approximately 5 kN/m^2 and less, at higher loads, the design method using partial restraints is found to predict higher strengths than are found in the laboratory. The strongest test result shown in figure 44 is for a wall that failed between 9 – 10 kN/m^2, yet the predicted strength was between 14 – 15 kN/m^2. It was for this reason that the design method adopted uses either full fixity or simple support conditions.



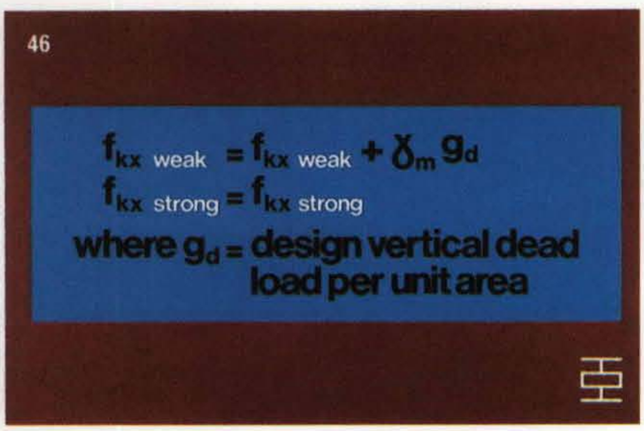
This approach[6], unlike partial restraint either estimates the strength realistically or *underestimates* the strength of the panels, as can be seen in figure 45, where all the points lie above line A – the 45° line. Line E represents the design method when using the partial safety factors for both the loads and the materials.

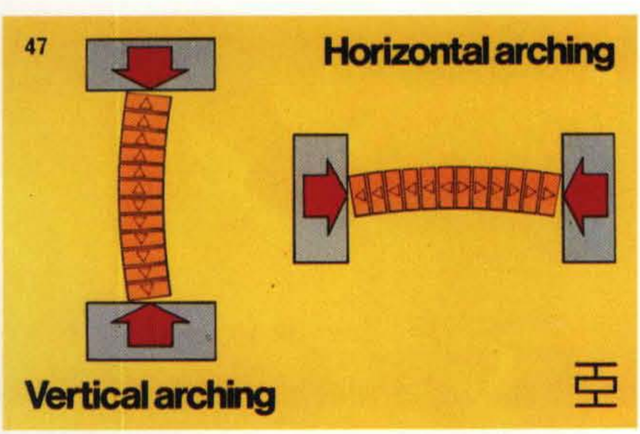
Allowing for precompression

In many cases, justifying a panel using the recommended design method is quite straightforward. In some cases, however, the panel's strength based on this method may be just inadequate for the load it is being designed to carry. In this situation, the characteristic flexural stress in the weak direction can be increased to allow for the dead weight of the top half of the panel. This enhances the strength of the panel by:

1. Increasing f_{kx} in the weak direction.
2. Modifying the α coefficient for 3 & 4-sided panels by changing the value of μ.

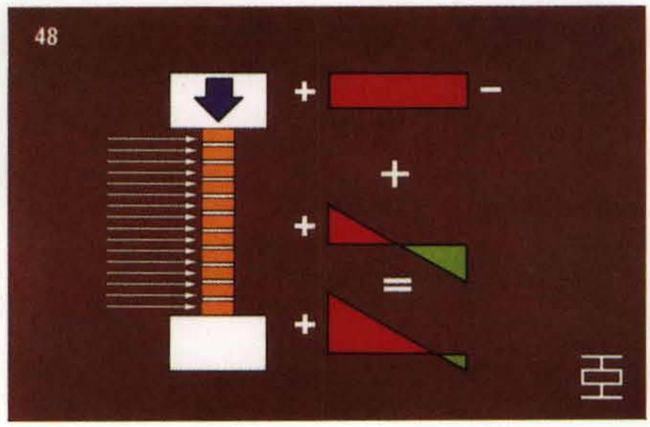
The strength enhancement is given by the formula in figure 46, below.



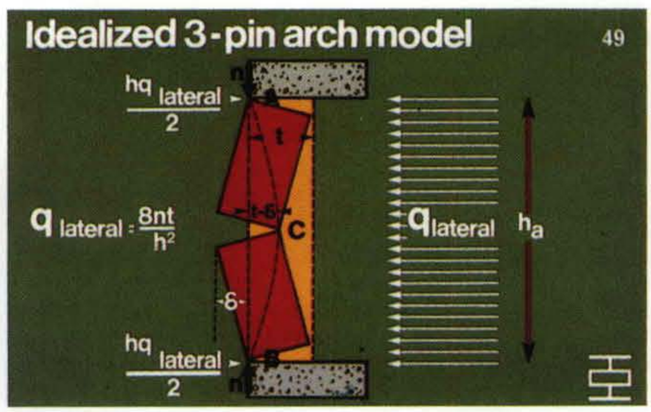


The design method outlined above has dealt with panels only. However, in situations where arching action can take place, this underestimates the strength of walls.

To accommodate the deflection induced by loading a wall panel, the panel must increase in length – either on plan or in section. If the wall is built solidly between rigid abutments, such as large concrete columns or substantial edge beams, it is not able to extend as it deflects. Instead, forces are generated within the plane of the wall, and they effectively induce restoring moments which stabilise the wall. Walls with arching action present are inherently much stronger than those – such as cladding panels – where arching action cannot take place. The Code gives design guidance on the strength of this type of wall.



Of particular interest is the case of the arching action of wind-loaded external walls in a loadbearing structure. In this case, a modified approach to the arching guidance given in the Code is suggested. In loadbearing walls, the arching force is generated from the precompressive stress already present due to the vertical load the wall is carrying.



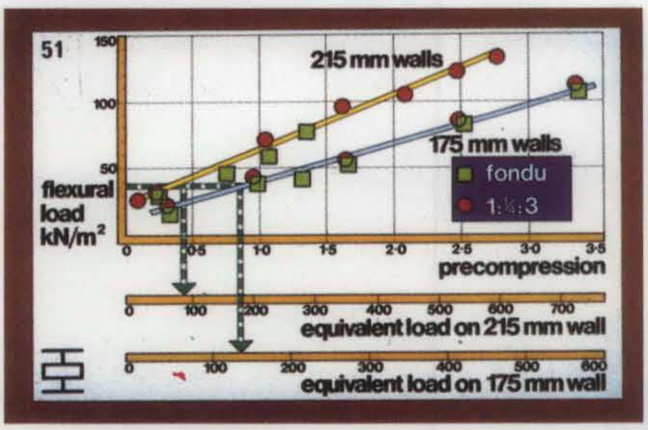
Even though a loadbearing wall cracks and forms three hinges at A, B and C, it does not necessarily fail. Failure will only occur when the lateral load reaches a level which will move the point C to the left of the line AB. In other words, when $\delta = t$.

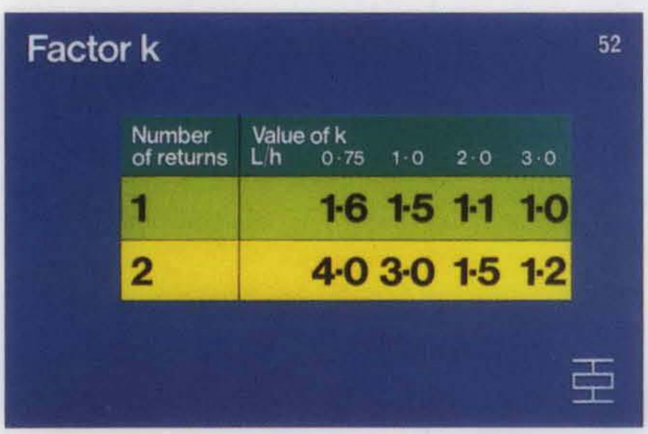
The basic equation $q_{lat} = \frac{8nt}{h^2}$ can be established by analysing the free body diagram of one half of the wall (figure 49). This equation is based on stability considerations and not on the flexural strength of the wall.

The equation was checked against the ultimate lateral load of wall panels in a full size 5-storey test building at the University of Edinburgh[9] (figure 50). The walls were removed by jacking and the failure loads ascertained. Further wall tests, conducted in laboratory machines under a constant precompression (figure 51 overleaf), confirmed the earlier conclusion that this ultimate load equation gives good correlation with the experimental results, and is a good failure prediction equation[10]. It is felt that the relatively small scatter associated with the body of test evidence is due to the stability nature of the equation.

The equation is for vertically spanning walls. Three and four-sided loadbearing walls have even greater lateral strength. To establish the lateral strength of a three or four-sided loadbearing wall, the strength is first calculated from the basic equation assuming the wall spans only vertically. The strength so obtained is then multiplied by an enhancement factor k, given in Table 10 of the Code (figure 52 overleaf).

The strength enhancement can be significant, 3 to 4 for square or nearly square aspect ratios. With larger aspect ratios,



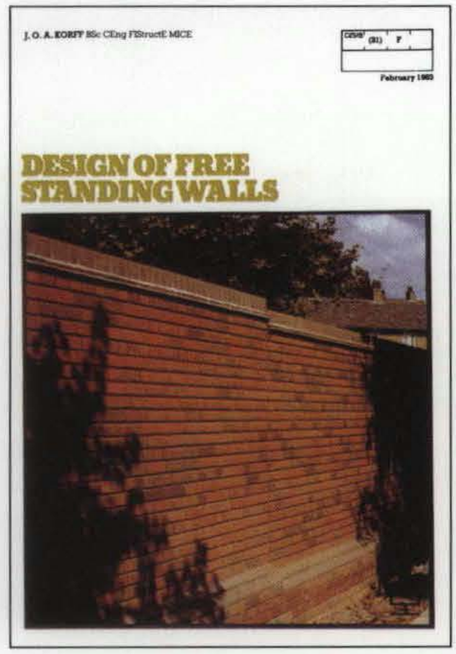


when the panel is three times as long as it is high, the stiffening effect of the vertical edge support diminishes to virtually zero and k approaches 1. Effectively, this recognises what could reasonably be predicted, namely, when L is much greater than h the panel tends to span purely vertically.

Freestanding walls

The Code gives guidance on the design of freestanding walls. The normal approach for brickwork freestanding walls is based on flexural strength using the modified flexural stress (for weak direction bending) mentioned earlier (page 11). f_{kx} is increased, as before, by the $\gamma_m g_d$ term. In the case of a freestanding wall, where the flexural failure is at the base, g_d is based on the dead weight of the *full* height of the wall.

A fuller treatment of all aspects of the design of freestanding walls, including details, materials specification, standard sections, strength design and the stability of footings is given in BDA Design Guide 12[11].



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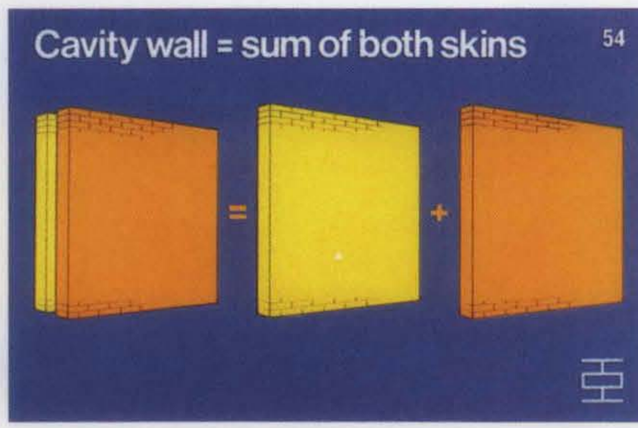
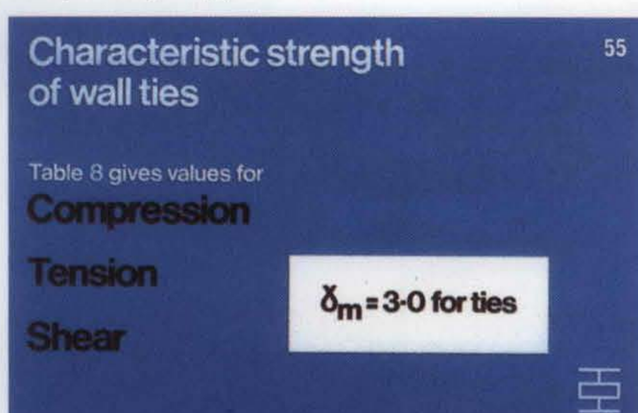
Cavity walls



Thus far, only the design of solid single-leaf walls has been discussed. In the case of loadbearing cavity walls, it is usually only the inner leaf which is analysed in arching, since this leaf normally carries the significant precompression. In cladding panel design, however, both leaves carry significant lateral load, and this must be taken into account.

Test results[6] indicate that the lateral strength of a cavity wall is equal to the sum of the strength of both individual leaves. This is a fortuitous finding and permits an easy and understandable design approach.

Wall panel edge supports



Whilst the method used to design laterally loaded walls against flexural failure has been covered, it is also important to ensure that the panel is adequately tied back to the structure. This is more critical with cladding panels than with loadbearing walls.

Cladding panels are usually tied back to their supporting structure using metal ties, and it is useful to have values for the strength of wall ties used as panel supports. Characteristic values for compression, shear and tension are given in Table 8 of the Code. When calculating design strengths, γ_m is 3 for ties.

Walls with openings

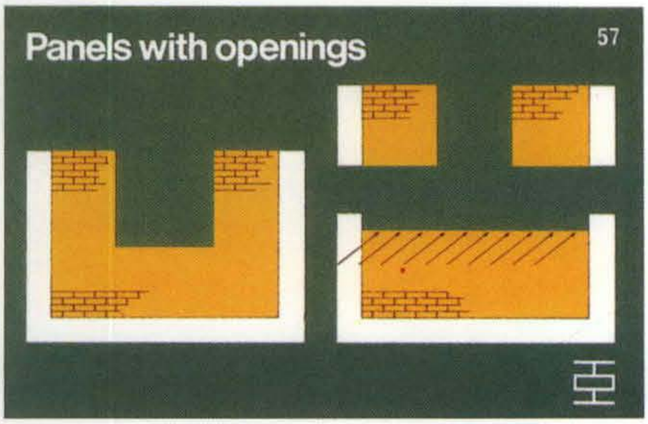

56

The design methods covered so far have dealt with single-leaf or cavity walls of solid construction with no door or window openings. The introduction of openings into panels reduces their ultimate lateral strength. Although some research has been conducted (figure 56) on perforate walls, no final conclusions have been reached. Consequently, no firm recommendations have been incorporated in the Code. Instead, in Appendix D, the Code recommends that either;

(a) a form of plate analysis is carried out:

or,

(b) a superimposition method is used (figure 57).


Panels with openings 57

A full treatment of panels with openings is outside the scope of this publication. The reader is referred to References 12 & 13.

The lateral load design of diaphragm and fin wall structures is not covered in the Code. Indeed, this subject is under review by the Code Technical Drafting Committee. While the theoretical basis of such designs follows normal sound engineering principles, as outlined above, the reader is referred to BDA Design Guides: 8 'Design of brick fin walls in tall single-storey buildings' and 11, 'Design of brick diaphragm walls', for more detailed guidance [14, 15].

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15. W.G. Curtin, G. Shaw, J.K. Beck & W.A. Bray. **Design of brick diaphragm walls.** Design Guide 11, Brick Development Association, March 1982.

16. J. Morton. **Accidental damage, robustness & stability.** Brick Development Association, May 1985.

Example 1

A cladding panel 2.6m high is to be designed in cavity brickwork. The wall has no returns, and because the vertical edges are unsupported, it spans vertically. The edge conditions at the base and the top of the wall are assumed to provide simple supports. The bricks to be used in both leaves have a water absorption of 9–9½% and the mortar is 1:1:6. Workmanship and materials are assumed to be normal category and the limiting dimensions are not exceeded.

Estimate the design wind pressure, W_k, which can be resisted by this panel.

The two basic equations are:

Design moment $= \alpha \gamma_f W_k L^2$

Design moment of resistance $= \dfrac{f_{kx} Z}{\gamma_m}$

For this panel:

$\alpha = 0.125$ for a simply supported case.

$\gamma_f = 1.2$ for a cladding panel whose failure does not affect the stability of the remaining structure.

$L = 2.6\text{m}$ span is taken as the height, h, *not* the length; since vertical spanning, use f_{kx} in the weak direction of bending.

$f_{kx} = 0.4\,\text{N/mm}^2$ for water absorption of 7–12% in 1:1:6 mortar.

$\gamma_m = 3.5$ for normal category construction control and manufacturing control of units.

The strength of a cavity wall is equal to the sum of the strengths of the two individual leaves.

For one leaf:

$$Z = 1000 \times \frac{102.5^2}{6} = 1.75 \times 10^6\,\text{mm}^3\text{/m run.}$$

$$M_d = \frac{0.4 \times 1.75 \times 10^6}{3.5} = \text{Nmm/m run} = 0.2\,\text{kNm/m run.}$$

$$M = 0.125 \times 1.2 \times W_k \times 2.6^2.$$

Equating M and M_d gives:

$$W_k = \frac{0.2}{0.125 \times 1.2 \times 2.6^2}\ \text{kN/m}^2 = 0.2\,\text{kN/m}^2$$

For cavity wall:

$$W_k = 0.2 + 0.2 = 0.4\,\text{kN/m}^2$$

This is a low value. W_k is unlikely to be as small as this in practice, and the panel could not be justified in many areas in Britain.

Example 2

With the above design example try to maximise W_k by taking the self-weight of the panel into account. Keep $\gamma_m = 3.5$ and use the same materials.

Characteristic dead load G_k (assume conservative, ie, low, values):

Outer leaf (102.5mm) $= 2.0\,\text{kN/m}^2$

Inner leaf (102.5mm) plastered one side $= 2.25\,\text{kN/m}^2$

The flexural strength in the weak direction can be modified to:

$f_{kx} + \gamma_m g_d$,

where g_d is the stress due to the design vertical load. The failure crack and therefore the critical section, ie, the position of maximum moment, is assumed to be at mid-height. Consider only the dead weight stress due to the top half of the wall.

Outer leaf:

Vertical load $= 1.3 \times 2.0\,\text{kN/m run}$

Design vertical load $= 0.9 \times 1.3 \times 2.0\,\text{kN/m run}$

Stress due to design vertical load $= \dfrac{2.34 \times 10^3}{102.5 \times 10^3}\ \text{N/mm}^2 = 0.023\,\text{N/mm}^2$

Modified flexural stress $= 0.4 + (3.5 \times 0.02)\,\text{N/mm}^2 = 0.47\,\text{N/mm}^2$

Basic equation $\alpha \gamma_f W_k L^2 = \dfrac{f_{kx} Z}{\gamma_m}$

$$0.125 \times 1.2 \times W_k \times 2.6^2 = \frac{0.47 \times 1.75}{3.5}$$

$$W_k = 0.23\,\text{kN/m}^2$$

Inner leaf:

Stress due to design vertical load $= \dfrac{0.9 \times 1.3 \times 2.25 \times 10^3}{102.5 \times 10^3}\ \text{N/mm}^2 = 0.026\,\text{N/mm}^2$

Basic equation $\alpha \gamma_f W_k L^2 = \dfrac{f_{kx} Z}{\gamma_m}$

$$0.125 \times 1.2 \times W_k \times 2.6^2 = \frac{[0.4 + (3.5 \times 0.026)] \times 1.75}{3.5}$$

$$W_k = 0.24\,\text{kN/m}^2$$

For the cavity wall, $W_k = 0.23 + 0.24\,\text{kN/m}^2 = 0.47\,\text{kN/m}^2$

The increase from 0.4 to 0.47 kN/m^2 is approximately 20%. This should be remembered when panels lie just outside justifiable conditions. (Note that the answer could have been achieved much quicker. Since f_{kx} was increased by approximately 20% when modified, W_k would increase also by approximately 20% because $W_k = K f_{kx}$ is a linear relationship.)

Example 3

A panel identical to that in Example 1 is to be constructed and tested in a laboratory. An estimate is needed, for rig design purposes, of the likely *ultimate* failure load. Calculate this value.

At present, the maximum *design* value, $W_k = 0.47$ kN/m^2. This value is derived using *design* values of γ_m, γ_f and *characteristic* values for flexural stress. For *ultimate* design $\gamma_m = \gamma_f = 1.0$ and the mean flexural stress values should be used.

Assuming no accurate values for the mean flexural stress are available, these can be estimated from figure 9. For bricks of 7–12% water absorption, the mean flexural stress can range between approximately 1.0 N/mm^2 and the 0.4 N/mm^2 f_{kx} value.

Assume $f_{mean} = 1.0$ N/mm^2 (strongest brick/mortar case).

Modifying this for self-weight (for simplicity, take $G_k = 2.5$ kN/m^2 for both leaves. 2.5 kN/m^2 is conservative, ie, high, for rig design):

$$f = [1 + \frac{1 \times 1.3 \times 2.5 \times 10^3}{102.5 \times 10^3}]$$
$$= 1.03 \text{ N/mm}^2$$

For one leaf:

$$0.125 \times 1.0 \times W_{max} \times 2.6^2 = \frac{1.03 \times 1.75}{1}$$
$$W_{max} = 2.13 \text{ kN/m}^2$$

For cavity wall: $W_{max} = 4.26$ kN/m^2

This is still not a true *maximum* value since it is based on a prediction of the maximum *mean* flexural stress. While the mean of, say, 10 wall tests should be approximately 4.26 kN/m^2 at lateral failure, the results will be scattered around this mean.

The example could be recalculated if the standard deviation associated with wallette test flexural strengths were known. However, this is outside the scope of this publication. Instead, the rig should be designed using 4.26 kN/m^2 together with an adequate factor of safety.

Example 4

A panel identical to that in Examples 1 & 2 is to be designed using different bricks and mortar. The material specification is now:

Outer leaf: water absorption < 7%; mortar 1:¼:3

Inner leaf: water absorption > 12%; mortar 1:¼:3

Calculate the maximum design wind pressure, W_k, taking account of the self-weight (assume $g_d = 2.25$ kN/m^2 for outer leaf, and 2.50 for inner leaf).

f_{kx} values: outer leaf 0.7 N/mm^2; inner leaf 0.4 N/mm^2.

Outer leaf:

$$0.125 \times 1.2 \times W_k \times 2.6^2 = \frac{[0.7 + (3.5 \times 0.026)] \times 1.75}{3.5}$$
$$W_k = 0.4 \text{ kN/m}^2$$

Inner leaf:

$$0.125 \times 1.2 \times W_k \times 2.6^2 = \frac{[0.4 + (3.5 \times 0.029)] \times 1.75}{3.5}$$
$$W_k = 0.25 \text{ kN/m}^2$$

For cavity wall: $W_k = 0.4 + 0.25$

$= 0.65$ kN/m^2

Hence, by changing the material specification, the design wind pressure W_k can be increased from 0.47 to 0.65 kN/m^2. Indeed, if the inner leaf were constructed using a brick of water absorption of less than 7%, W_k could be further increased to approximately 0.8 kN/m^2. This is adequate for many situations in Britain.

Example 5

Compare the design wind pressure $W_k = 0.65$ kN/m^2, from the previous example, with the design wind pressure for a wall, using the same brick and mortar materials, which is 2.6 m *long* and purely horizontally spanning (ie, assume the base of the wall is a free edge).

f_{kx} values: outer leaf = 2.0 N/mm^2; inner leaf = 1.1 N/mm^2.

Outer leaf:

$$0.125 \times 1.2 \times W_k \times 2.6^2 = 2 \times \frac{1.75}{3.5} : W_k = 0.99 \text{ kN/m}^2$$

Inner leaf:

$$0.125 \times 1.2 \times W_k \times 2.6^2 = 1.1 \times \frac{1.75}{3.5} : W_k = 0.54 \text{ kN/m}^2$$

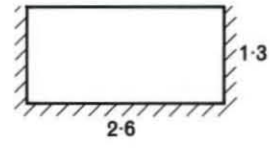
For cavity wall: $W_k = 0.99 + 0.54$ kN/m^2

$= 1.53$ kN/m^2

This is significantly stronger than the 0.65 kN/m^2 derived in Example 4 and demonstrates the advantage of arranging panel support conditions in a way which encourages panels to bend in their strong horizontal direction. The increase in strength is solely due to this. A design wind pressure of 1.53 kN/m^2 is more than adequate for virtually all conditions in Britain. (Note: no dead weight was used to enhance f_{kx}; this is only appropriate for weak direction bending.)

Example 6

Repeat Example 5 for the practical case where the wall is supported at its base. Assume the base of the wall is simply supported, since it will be sitting on dpc material. The height of the panel is 1.3 m. No loads are taken on the panel other than the wind loading its surface.



Length = 2.6 m, height = 1.3 m.

$h/L = 0.5$; $\mu = 0.35$

$\alpha = 0.064$, Table 9 (A), BS 5628

Outer leaf:

$$0.064 \times 1.2 \times W_k \times 2.6^2 = 2 \times \frac{1.75}{3.5}, \; W_{ko} = 1.93$$

Inner leaf:

$0.064 \times 1.2 \times W_k \times 2.6^2 \;=\; 1.1 \times \frac{1.75}{3.5},\;\; W_{ki} = \frac{1.06}{2.99}\,kN/m^2$

This is adequate for anywhere in Britain.

Example 7

A cladding panel is subject to a design wind pressure, W_k, of 0.8 kN/m² (suction). It is a corner panel, see below, free at the top edge but sitting on dpc material. If the wall is to be designed using 102.5 mm brickwork in both leaves, are there any limitations on the specification of the bricks or the mortar?

Figure A

The assumed edge conditions are shown below.

The continuity of the outer leaf over the column and around the corner is asumed to provide fixity to the vertical edges.

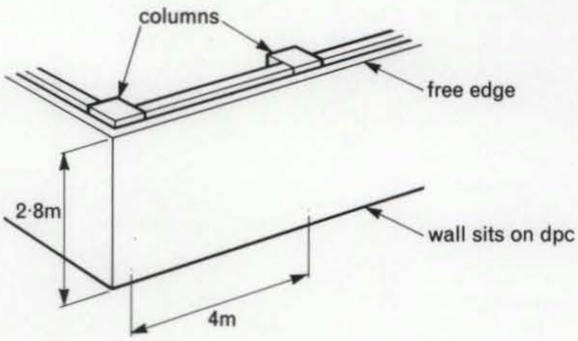
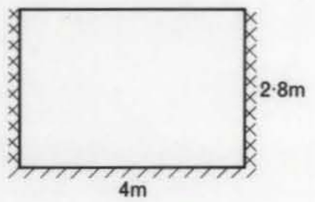


Figure B

Limiting dimensions (BS 5628, Cl.36.3):

$t_{ef} = 2/3\,(102.5 + 102.5)$	= 137 mm
Maximum area $\not> 1500 t_{ef}^2$	= 28 m²
Actual area	= 11.2 m²
Maximum dimension $\not> 50\,t_{ef}$	= 6.85 m
Actual largest dimension	= 4 m



Since neither the maximum area nor the maximum dimension is exceeded, the panel satisfies the limiting dimension requirements.

Basic equation for one leaf:

$\alpha \gamma_f W_k L^2 \text{ per leaf} \;=\; \frac{f_{kx} Z}{\gamma_m}$

$\gamma_f = 1.2$; $W_k = 0.4\,kN/m^2\,(W_{cav} = W_1 + W_2$, but $W_1 = W_2)$; $\gamma_m = 3.5$; $Z = 1.75 \times 10^6\,mm^3/m$.

α coefficient: $\frac{h}{L} = \frac{2.8}{4} = 0.7;\; \mu = 0.35.$

$\alpha \;=\; 0.039 \text{ for } h/L = 0.5$
$\;\;\;=\; 0.045 \text{ for } h/L = 0.75$ } BS 5628, Table 9 (C)

therefore, $\alpha \;=\; 0.044 \text{ for } h/L = 0.7$, by interpolation.

$M \;=\alpha\gamma_f W_k L^2 \;=\; 0.044 \times 1.2 \times 0.4 \times 4^2\,kNm/m \text{ run}$

$\;=\; 0.338\,kNm/m \text{ run}$

$\;=\; 0.388 \times 10^6\,Nmm/m \text{ run}.$

$M_d = f_{kx}\frac{1.75 \times 10^6}{3.5}\,Nmm/m \text{ run}.$

Equating M and M_d gives: $f_{kx}\frac{1.75}{3.5} \times 10^6 = 0.338 \times 10^6.$

f_{kx} reqd. = 0.68 N/mm² (strong direction).

Clay bricks

The weakest f_{kx} value in Table 3 is 0.8 N/mm². Since this exceeds the required value, any clay brick/mortar combination can be used.

Calcium silicate bricks

The two values of f_{kx} given in Table 3, in the strong direction, are 0.9 and 0.6 N/mm². Only the 0.9 N/mm² is acceptable (ie, mortar designations (i), (ii) & (iii)). It is normal, when using calcium silicate brickwork, for a mortar no stronger than designation (iii) to be used. Hence, specification would probably be 'any calcium silicate brick using a 1:1:6 mortar (or equivalent)'.

Example 8

Because the panel just designed in Example 7 is in a sheltered position, the architect would like to use a calcium silicate brick in a 1:2:9 mortar (designation (iv)). Can this be justified?

The ways in which the panel can be strengthened and possibly justified are:

(i) modify the flexural strength using the panel's self-weight;

or,

(ii) detail the top edge of the panel to give a simple support; or,

(iii) use horizontal bed joint reinforcement in one or both leaves.

Since option (iii) is outside the scope of this publication, option (i) will be explored – only a small reduction (approximately 10%) is needed in the f_{kx} required. If the panel cannot be justified, option (ii) will be considered.

Option (i)

Assume:

Dead load of both leaves = 2.0 kN/m² (low value of G_k for conservative reason)

Vertical load $= \frac{2.8}{2} \times 2.0 \;= 2.8\,kN/m$

Design vertical load $= 0.9 \times 2.8 \quad = 2.5\,kN/m$

Stress due to design vertical load $= \dfrac{2.5 \times 10^3}{102.5 \times 10^3} = 0.025\,N/mm^2$

Assume a calcium silicate brick using 1:2:9 mortar (designation (iv)). Modifying f_{kx} (this is done in the *weak* direction):

f_{kx} (strong) $= 0.6\,N/mm^2$;
f_{kx} (weak) $= 0.2 + 3.5 \times 0.025$
$= 0.288\,N/mm^2$

For a single leaf:
$M = \alpha\gamma_f W_k L^2$
α coefficient, from Table 9 (C), $\mu = 0.5$:
$\alpha = 0.035$; $h/L = 0.5$
$\alpha = 0.043$; $h/L = 0.75$
$\alpha = 0.041$; $h/L = 0.70$, by interpolation.
$M = 0.041 \times 1.2 \times W_k \times 4^2$
$= 0.79\,W_k$ kNm/m run

$$M_d = \frac{f_{kx} Z}{\gamma_m} = \frac{0.6 \times 1.75 \times 10^6}{3.5 \times 10^6}$$

$= 0.3$ kNm/m run
Equating M and M_d

$$W_k = \frac{0.3}{0.79} = 0.38\,kN/m^2.$$

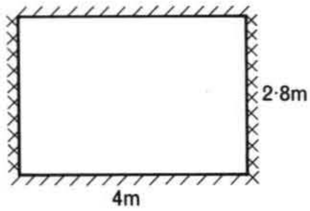
For cavity wall:
$W_k = 2 \times 0.38 = 0.76\,kN/m^2$.
This is less than $W_k = 0.8\,kN/m^2$.

Option (ii)
If calcium silicate brickwork were to be used in conjunction with a 1:2:9 (designation (iv)) mortar, it would be advisable to support the top of the wall in some way. Normally, this is done using either special (sliding) fixings in conjunction with special wall ties, or by using sections of angle or channel to restrict the lateral movement of the top edge. If this were done with suitable details the panel would be four-sided.

Checking the panel, but not taking account of self-weight, and repeating the earlier section for one leaf:

$$M_d = f_{kx}\frac{1.75 \times 10^6}{3.5}\ \text{Nmm/m run.}$$

Figure C

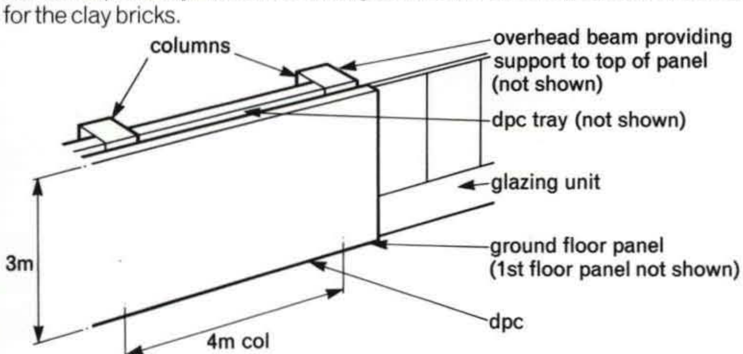


$M = \alpha\gamma_f W_k L^2$.
α coefficient:
$h/L = 0.7$; $\mu = 0.35$;
interpolation from Table 9 (G) gives $\alpha = 0.031$.
$M = 0.031 \times 1.2 \times 0.4 \times 4^2 = 0.238$ kNm/m run.
Equating M and M_d gives f_{kx} reqd. $= 0.48\,N/mm^2$.

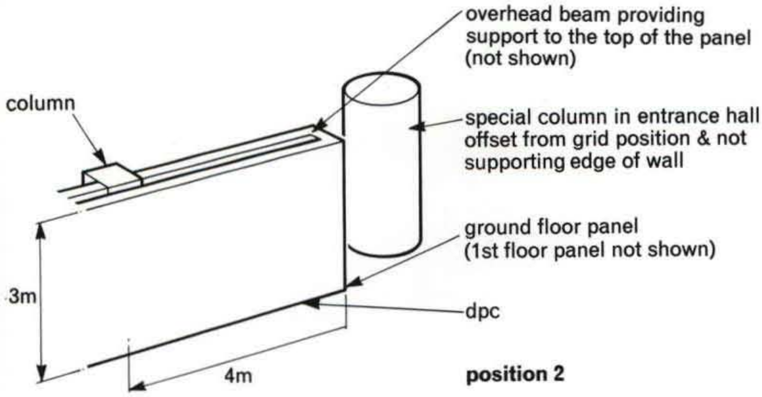
This is less than the f_{kx} value of $0.6\,N/mm^2$ in Table 3. So, the panel works satisfactorily as a four-sided panel in calcium silicate brickwork in a 1:2:9 or equivalent designation (iv) mortar.

Example 9

A similar panel to Example 7 is to be designed for two different parts of the structure. The W_k value for both positions (general wind pressure) is $0.68\,kN/m^2$. It is proposed to use a 100 mm $3.5\,N/mm^2$ block and a clay brick (both in 1:1:6 mortar) for the inner and outer leaves respectively. If the two arrangements for the two positions are as shown below, comment on any specification requirements for the clay bricks.

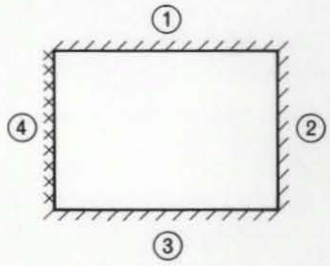


position 1



Position 1

Assumed edge support conditions:
Side 1 simple support, dpc tray above
Side 2 simple support, panel stops – but provide support ties
Side 3 simple support, sits on dpc.
Side 4 fully fixed, front leaf continues past column.



Note: Clause 36.2 permits both leaves to be assumed continuous, even if only one leaf is continuous, if (i) cavity wall ties are used in accordance with Table 6, (ii) the discontinuous leaf is not thicker than the one that is continuous. Hence, the assumed edge conditions shown above apply to both leaves.

Inner leaf:

100 mm 3.5 N/mm² block in 1:1:6 mortar.
f_{kx} weak = 0.025 N/mm²; f_{kx} strong = 0.45 N/mm².
$\mu = 0.025/0.045 = 0.55$; $h/L = 3/4 = 0.75$.
From Table 9 (F):

$$\alpha = 0.034;\ Z = 1000 \times \frac{100^2}{6} = 1.67 \times 10^6 \text{ mm}^3\text{/m run.}$$

$$\text{Equating M and } M_d,\ \alpha\gamma_f W_k L^2 = \frac{f_{kx} Z}{\gamma_m}$$

$$0.034 \times 1.2 \times W_k \times 4^2 = \frac{0.45 \times 1.67 \times 10^6}{3.5 \times 10^6}$$

$$W_k = 0.33 \text{ kN/m}^2.$$

Since the cavity wall must carry 0.68 kN/m², the outer leaf must carry 0.68 − 0.33 = 0.35 kN/m².

Outer leaf:

Assume the weakest case in 1:1:6 mortar and check the W_k found against the 0.35 kN/m² required to be carried:
α = 0.041 (μ = 0.35; h/L = 0.75; Table 9 (F))
f_{kx} = 0.9 N/mm².

$$\text{Equating M and } M_d,\ \alpha\gamma_f W_k L^2 = \frac{f_{kx} Z}{\gamma_m}$$

$$0.041 \times 1.2 \times W_k \times 4^2 = \frac{0.9 \times 1.75 \times 10^6}{3.5 \times 10^6}$$

$$W_k = 0.57 \text{ kN/m}^2.$$

This compares favourably with the 0.35 kN/m² the outer leaf will be required to carry if the inner leaf was fully worked.

There are no specification requirements for the clay bricks of the panel in Position 1. Any clay brick in 1:1:6 mortar will be sufficient. (Design load = 0.68 kN/m², Design resistance = 0.9 kN/m².)

Position 2

Assumed edge support conditions:
Same as above (Position 1)
except that edge 2 is free.

Repeating the above calculations (in large steps) but basing the α coefficients on Table 9 (K).

Inner leaf:

$$\alpha\gamma_f W_k L^2 = \frac{f_{kx} Z}{\gamma_m}$$

$$0.057 \times 1.2 \times W_k \times 4^2 = \frac{0.45 \times 1.67}{3.5}$$

$$W_k = 0.20 \text{ kN/m}^2.$$

Outer leaf:

$$0.075 \times 1.2 \times W_k \times 4^2 = \frac{0.9 \times 1.75}{3.5}$$

$$W_k = 0.31 \text{ kN/m}^2.$$

Cavity wall:

Design wind resistance = 0.20 + 0.31 = 0.51 kN/m².

This is not adequate, and a different specification will be required.

Either,
recalculate using an f_{kx} of 1.5 N/mm² (to give W_k = 0.52 kN/m²)
or,
on the basis that W_k varies linearly with f_{kx},

$$W_k = \frac{0.31 \times 1.5}{0.9} = 0.52 \text{ kN/m}^2 \text{ when } f_{kx} = 1.5 \text{ N/mm}^2.$$

Thus, design wind resistance is now 0.20 + 0.52 = 0.72 kN/m². This exceeds the design wind pressure of 0.68 kN/m². Therefore, in position 2, the clay bricks need to be of less than 7% water absorption in a 1:1:6 or equivalent designation (iii) mortar.

(Note: It would be quite possible to investigate the strength of the outer leaf using a brick of 12% water absorption or greater in a stronger mortar. It is unusual, however, to detail a cavity wall with different mortars in each leaf; it would require careful supervision in practice.)

If it is planned to use bricks with a water absorption higher than 7%, the strength of the inner leaf will require to be enhanced. This could be achieved by:

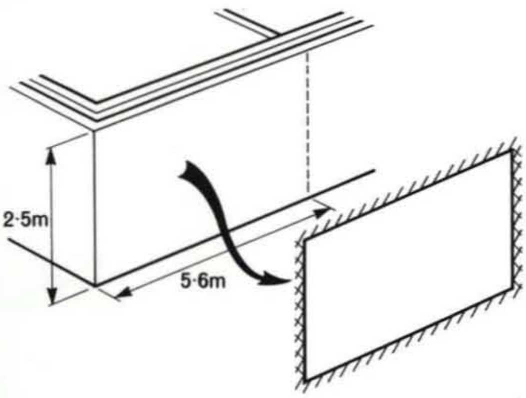
(i) Increasing the block strength specified.
(ii) Increasing the thickness of the inner leaf. This would increase Z.

(iii) Reinforcing the blockwork.

Reinforcing the outer or inner leaf (or both) may be the only solution if a brick of more than 12% water absorption is to be used.

The above options assume that the simplest solution – namely, providing edge support to the assumed free edge – cannot be achieved. This solution should, of course, be explored first.

Example 10



A loadbearing cavity wall is to be checked for lateral strength. It is well supported on its two vertical edges, but is assumed to be simply supported top and bottom. Both leaves carry the roof equally. W_k for this exposed site is 1.2 kN/m^2 suction.

Roof loads:

Looking for conservative (unfavourable) conditions, and using dead + wind (no imposed) based on 0.9 G_k and 1.4 W_k, the minimum design vertical load is 6.2 kN/m of wall, ie, 3.1 kN/m on each leaf. Both leaves have bricks of more than 12% water absorprtion in 1:¼:3 mortar. f_{kx} (weak) = 0.4 N/mm^2; f_{kx} (strong) = 1.1 N/mm^2; (assume $\mu = 0.35$).

Stress due to design vertical load $= \dfrac{3.1 \times 10^6}{102.5 \times 10^6} = 0.03$ N/mm^2

Stress due to design self-weight of wall $= \dfrac{0.9 \times 1.25 \times 2.0 \times 10^3}{102.5 \times 10^3} = 0.02$ N/mm^2

Total stress for design loads $= \overline{0.05}$ N/mm^2

f_{kx} (weak) $= 0.4 + 3.5 \times 0.05 = 0.58$ N/mm^2; $\mu = 0.58/1.1 = 0.52$.

First check whether it will work as cladding (ie, no allowance for vertical load):

$h/L = 0.45$; $\alpha = 0.022$ (Table 9 (G): $\mu = 0.35$).

$$\alpha \gamma_f W_k L^2 = \frac{f_{kx} Z}{\gamma_m}$$

$$0.022 \times 1.4 \times W_k \times 5.6^2 = \frac{1.1 \times 1.75}{3.5}; \ W_k = 0.57 \text{ kN/m}^2.$$

Cavity wall design strength resistance = 1.14 kN/m^2; design load, W_k = 1.2 kN/m^2.

Accounting for design vertical stress:

$\alpha = 0.018$; $\mu = 0.52$; $h/L = 0.45$

$$0.018 \times 1.4 \times W_k \times 5.6^2 = \frac{1.1 \times 1.75}{3.5}$$

$$W_k = 0.70 \text{ kN/m}^2$$

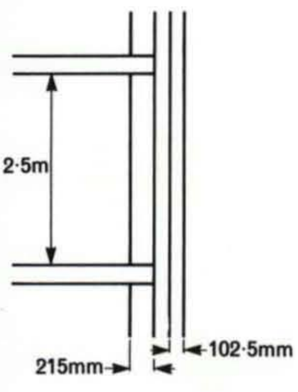
Cavity wall design strength resistance = 1.40 kN/m^2.

This is adequate for W_k = 1.2 kN/m^2.

Example 11

(To be read in conjunction with BDA publication 'Accidental Damage, Robustness & Stability'[16])

A 215mm brickwork wall is to be checked to ascertain whether it can withstand the 'accidental force' equivalent load of 34 kN/m^2 (the equivalent gas explosion lateral pressure). The design load on the wall is 140 kN/m run (derived from using partial load factors appropriate for accidental damage analysis – Clause 22 (d)).



The basic equation $q_{lat} = \dfrac{8nt}{h^2}$ is used for walls with sufficient precompression to develop in-plane (ie, arching) forces. For accidental damage analysis using this equation a partial factor of safety for materials represented by $\gamma_m = 1.05$ can be used (Clause 37.1.1).

$$q_{lat} = \frac{8 \times 0.215 \times 140}{1.05 \times 2.5^2}$$
$$= 36.7\ kN/m^2.$$

Since this is in excess of 34 kN/m^2, the wall is judged to remain after an 'accidental event' and is regarded as a protected member (Clause 37.1.1).

Repeating the design, but using t = 170 mm,

$$q_{lat} = \frac{8 \times 0.170 \times 140}{1.05 \times 2.5^2}$$
$$= 29\ kN/m^2\ (cf.\ 34\ kN/m^2).$$

If the wall was constructed using the 170 mm Calculon brick, it would be judged to be removed by an accidental event.

(Note that, if there were one or two returns on the vertical edge(s), the values of 36.7 and 29 kN/m², above, should be enhanced by the appropriate k factor from Table 10.)

Assuming that it was decided not to use a 215 mm, one brick thick, inner loadbearing leaf, the building would need:
either,
(i) to be checked to make sure a partial collapse of unacceptable proportions did not ensue;
or,
(ii) to be fully horizontally and vertically tied.
See Clause 37.1 and Table 12.

For a fuller explanation of the subject of accidental analysis, see 'Accidental Damage, Robustness & Stability'[16].

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