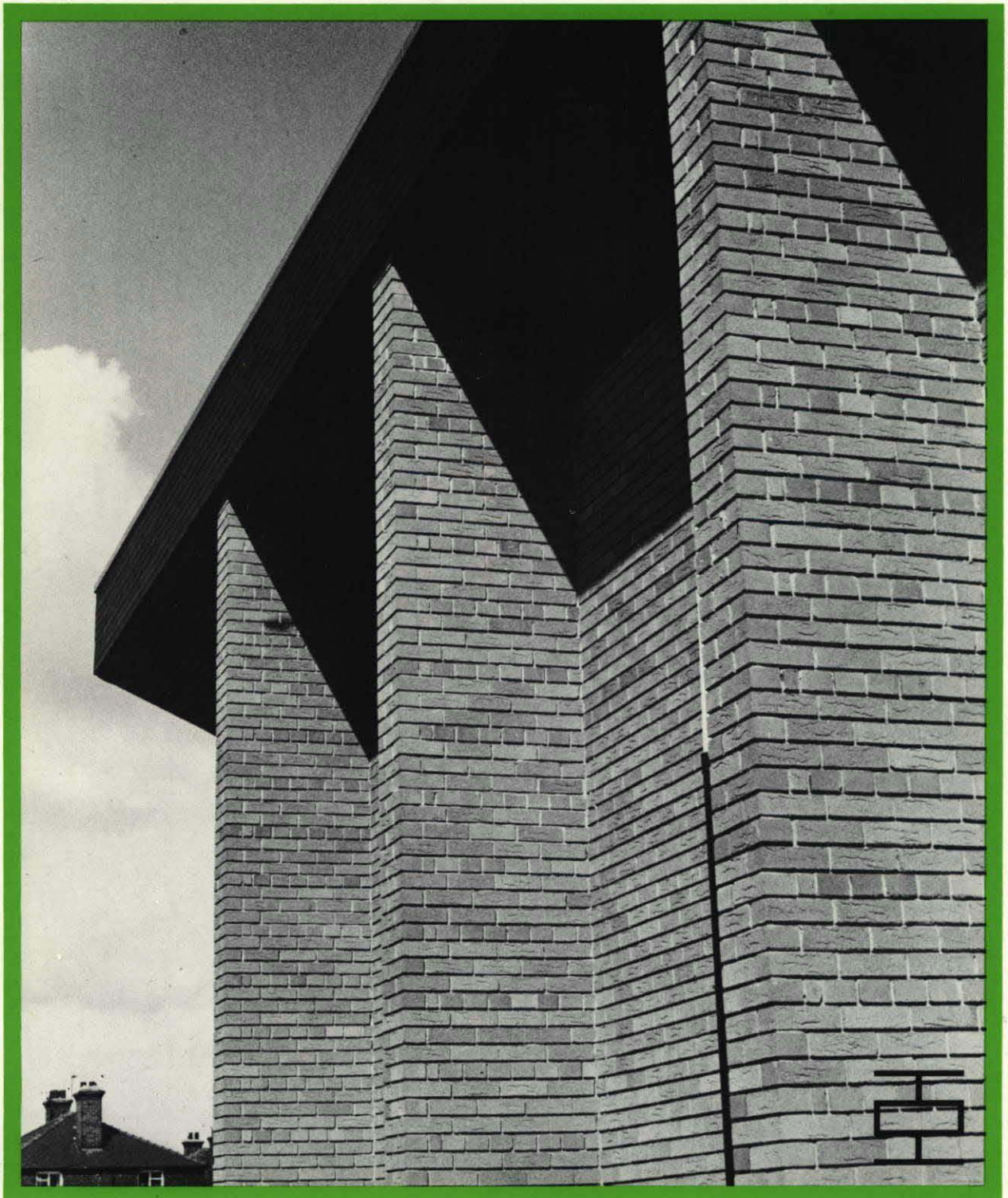


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June 1980

DESIGN OF BRICK FIN WALLS IN TALL SINGLE -STOREY BUILDINGS



Price £3.00

Design of brick fin walls in tall single-storey buildings

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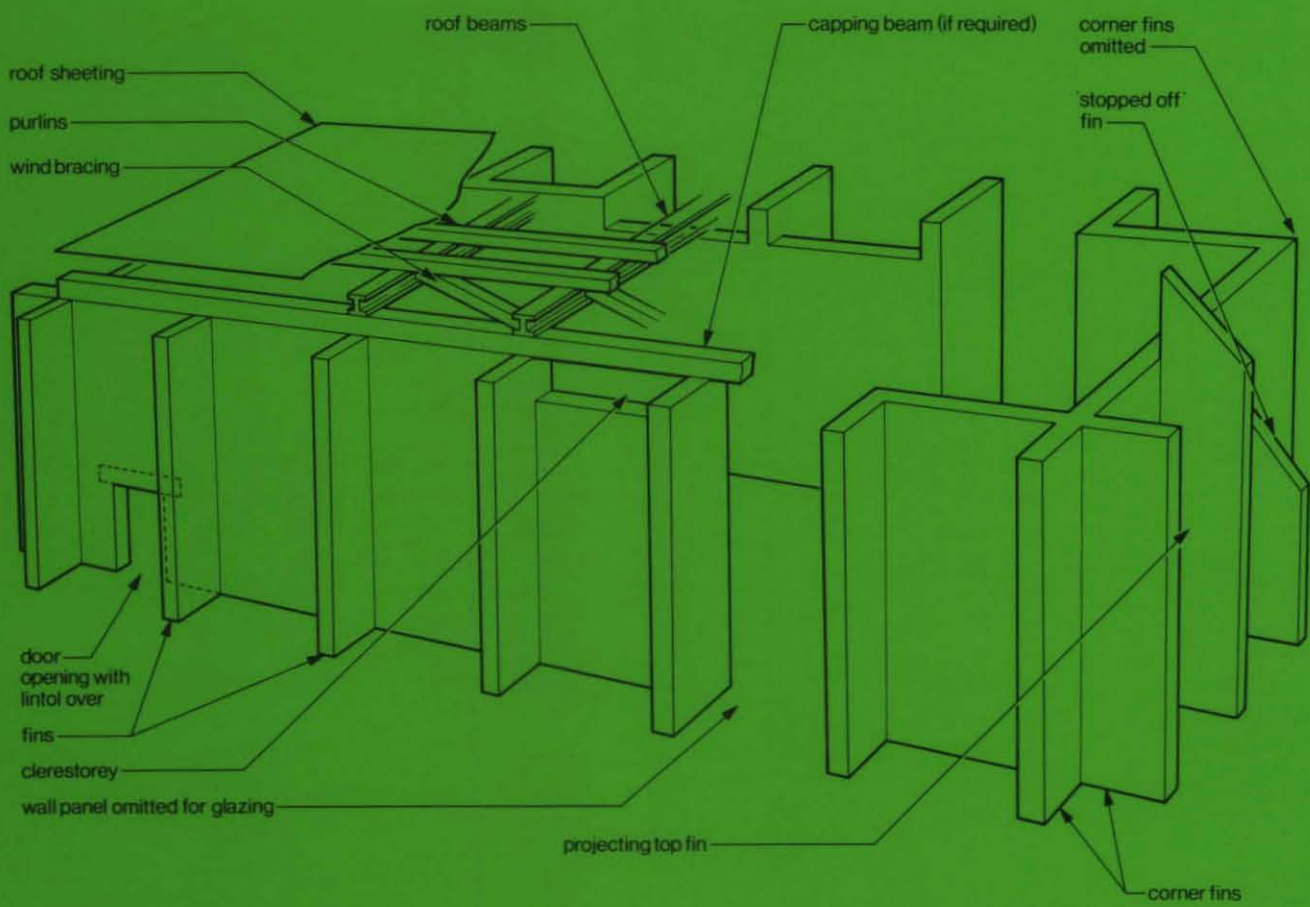
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INTRODUCTION AND GENERAL DESIGN

INTRODUCTION

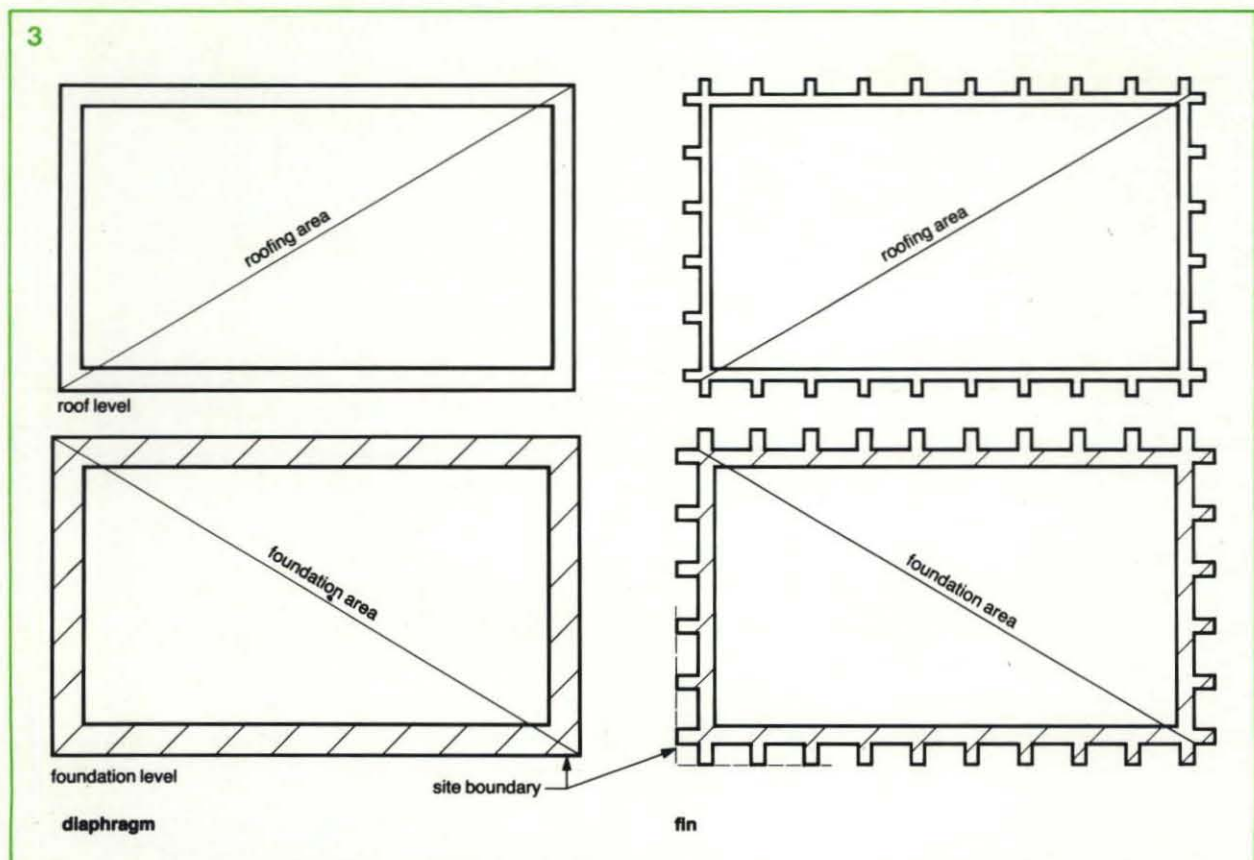
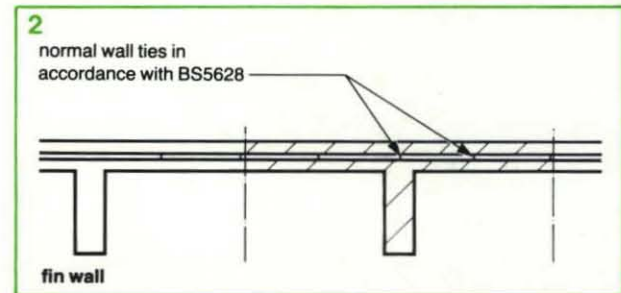
The fin wall, in which deep piers or fins are used to support conventional cavity walls has been shown by some considerable experience to be particularly suited, as are diaphragm walls, for tall single-storey wide span buildings such as assembly and sports halls, gymnasiums, swimming pools, industrial buildings, etc.

Fin wall buildings are another type of masonry structure that obviate the need for steel or reinforced concrete columns, external cladding and internal lining.

The brickwork T section formed (Fig 2) by the fin and outer leaf of the cavity wall provides the main structural member, and the inner leaf is the internal lining. Thus, only one material used by one trade under the direct control of the general

contractor is needed to form a durable, maintenance free, attractive and economical wall.

The fin wall was developed from the diaphragm wall⁽¹⁾ when an architect wished to maintain a cavity and have greater scope for architectural expression. The width of the cavity may be selected within the limitations of the code, BS 5628⁽²⁾, to allow for the use of cavity insulation.



And, if necessary, the thickness of the outer leaf may be increased to allow wider spacing of fins.

Comparison of fin walls and diaphragm walls

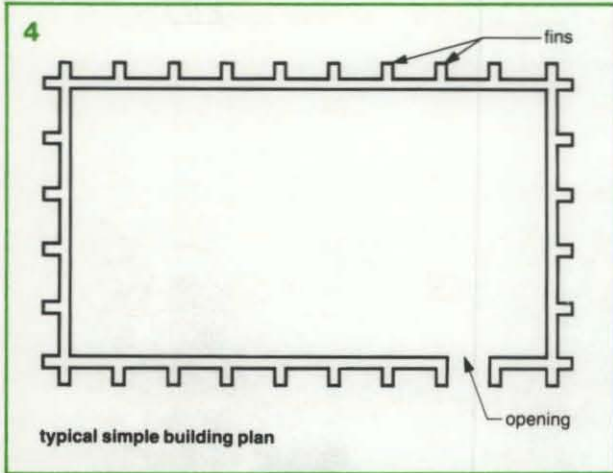
Advantages of fin walls over diaphragm walls:

- (1) Less roof area is required (see Fig 3)
- (2) Less foundation area is required (see Fig 3)
- (3) There is generally less cutting of bricks for bonding
- (4) There is obvious scope for architectural effect
- (5) They are easier to post-tension when required

(6) They have greater potential for use in multi-storey structures.

Disadvantages of fin walls compared with diaphragm walls:

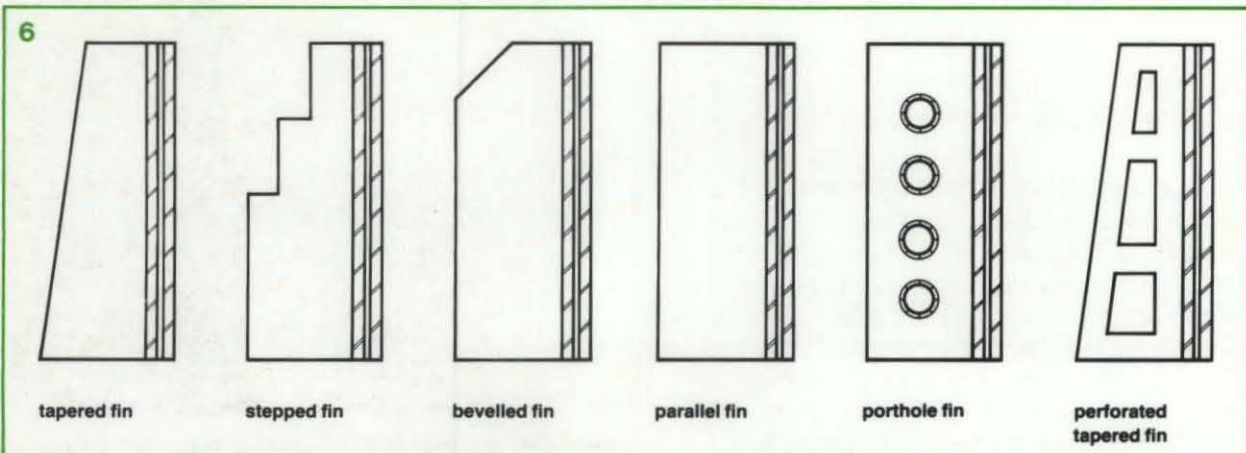
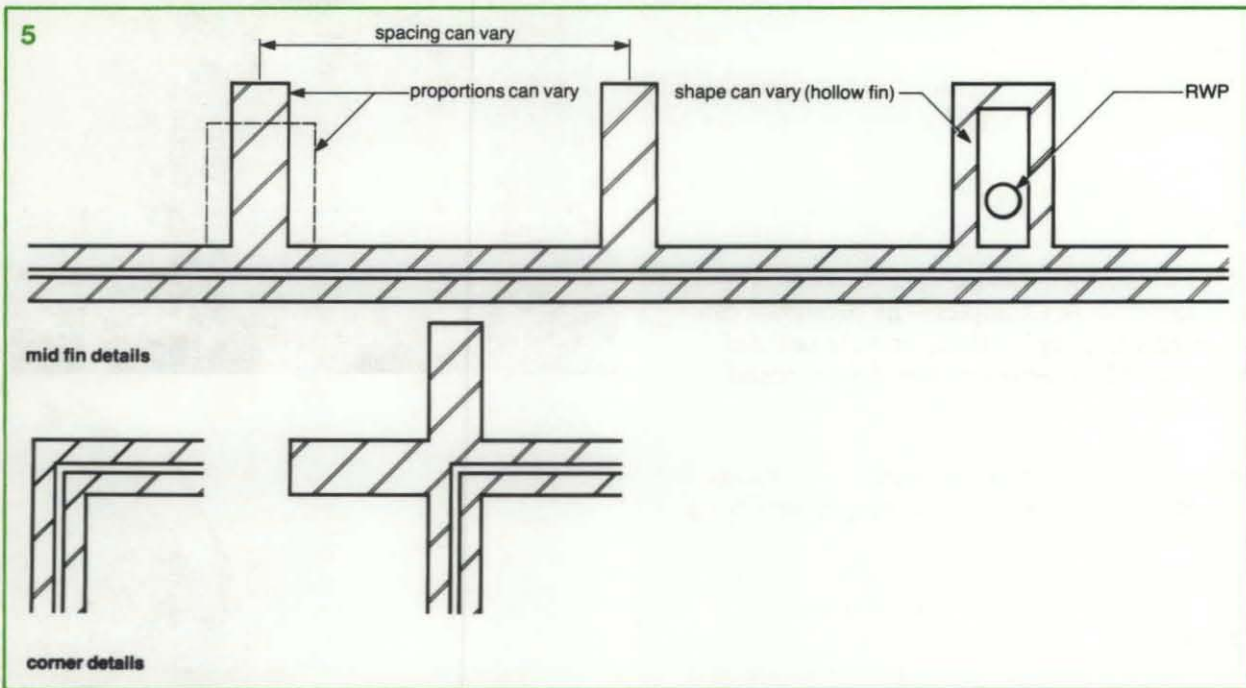
- (1) A larger site area is required (see Fig 3), so they may not be viable for restricted sites
- (2) The non-symmetrical section does not have a similar resistance to bending in both directions and a slightly greater cross-sectional area of brickwork is sometimes required
- (3) The cavity is not wide enough to accommodate services
- (4) More vertical plumbing lines (ie. corners of fins and fin wall connections) are required, so the labour costs of the wall tend to be higher.

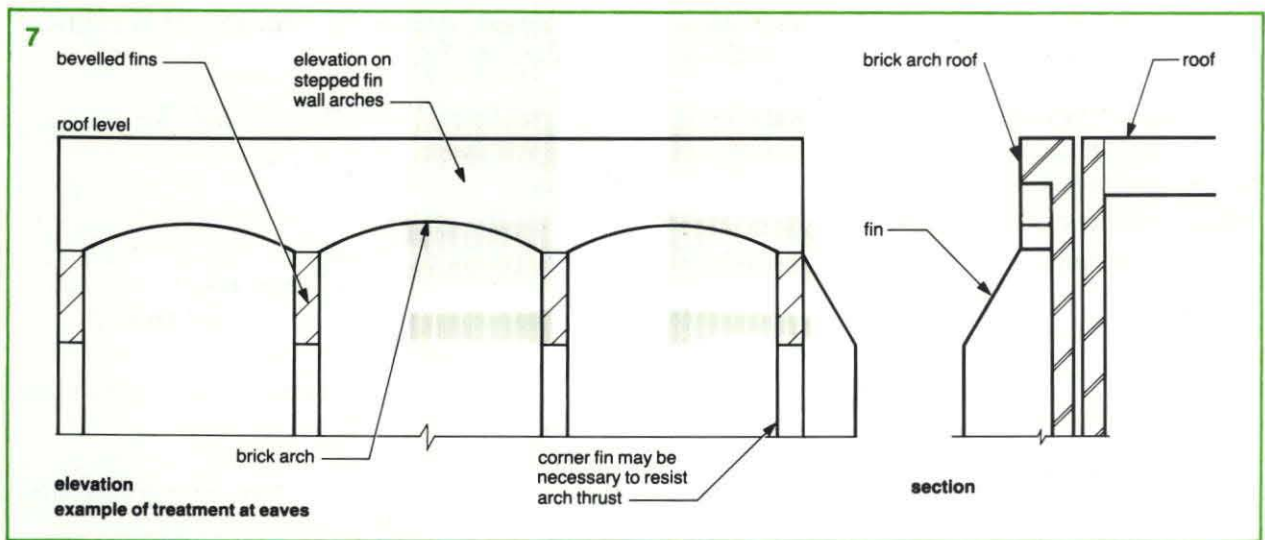


GENERAL DESIGN

Structural principles

In the current codes of practice, piers are recognised as a means of increasing the vertical load-carrying capacity of plane walls. The effect of piers is presented as an apparent increase in the effective thickness of the plane wall. This, of course, will give a lower slenderness ratio and thus the wall can be allowed to carry a larger vertical load. In CP 111⁽³⁾ no specific guidance





was given for the design of walls with piers under lateral load. BS 5628 gives some guidance.

In the concept of the fin wall given in this guide, the whole fin plus the wall is used in determining the slenderness ratio although, as with diaphragm walls, for tall single-storey buildings, vertical loading is unlikely to be critical. The critical loading case for designing such walls will generally be that of wind loads and for this a T section is considered.

Architectural treatment

A typical simple plan layout for a rectangular building is shown in Fig. 4.

Examples of the variations which can be made to this plan are that the sizes and spacing of the fins, the details at the corners and even the basic wall profile can be varied in many different ways. Fig 5 shows some examples. The variations can have structural implications, and the selected profile must be checked to suit the structural needs.

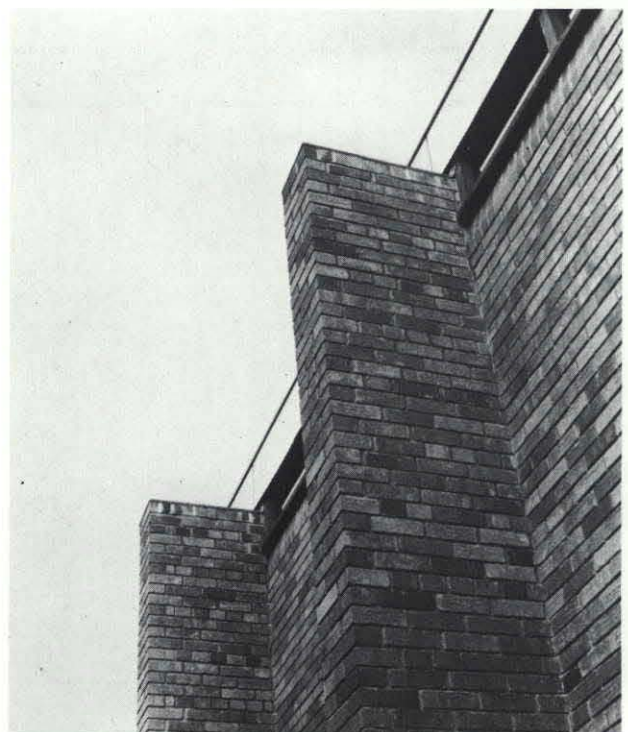
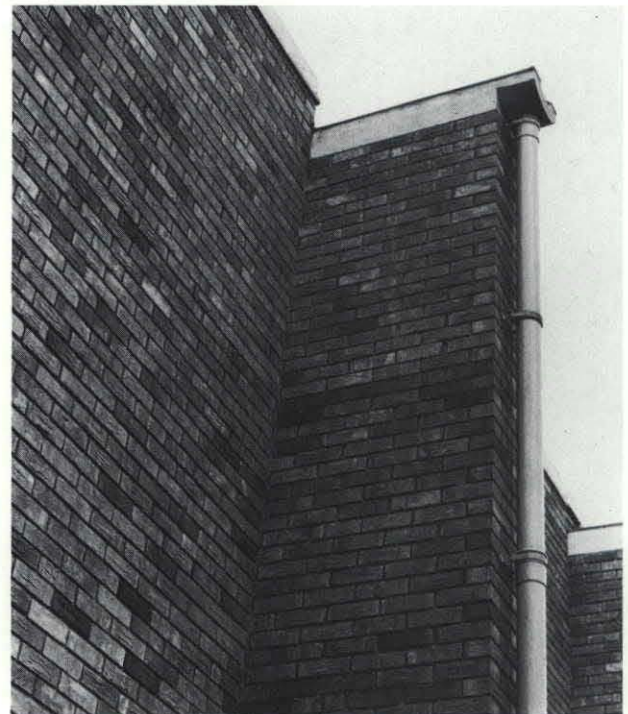
On elevation, the fin can be tapered, bevelled, profiled, etc. and some typical shapes are shown in Fig 6.

The treatment at the eaves of the roof, the variety and mixtures of bricks, and the type of fin gives the construction many possible aesthetic effects (see Fig 7). Thus, the designer has great scope for architectural treatment.

Some typical examples of fin wall construction are shown in the illustrations to this section.

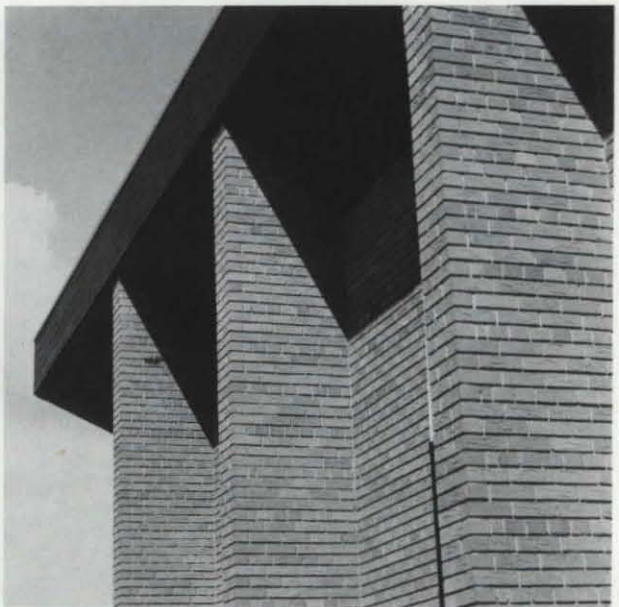
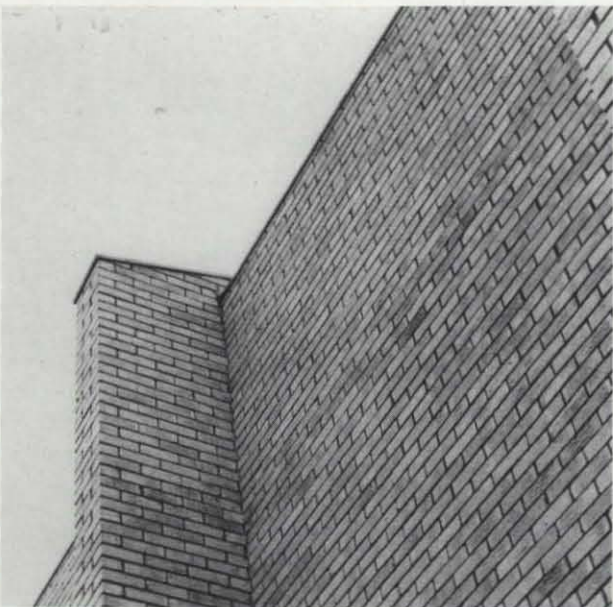
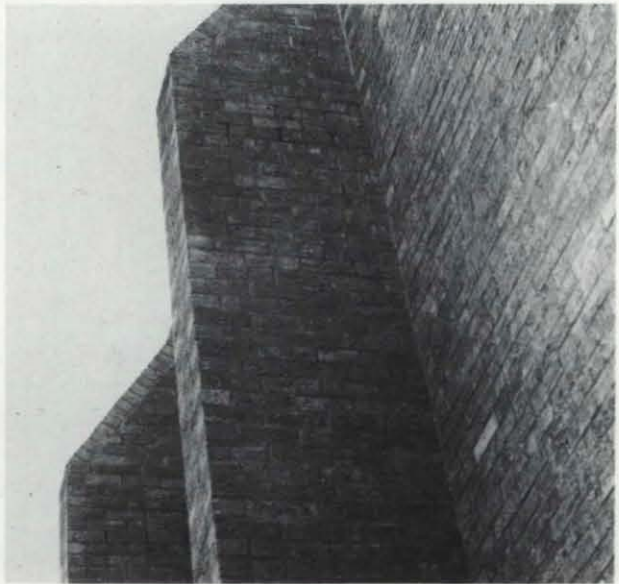
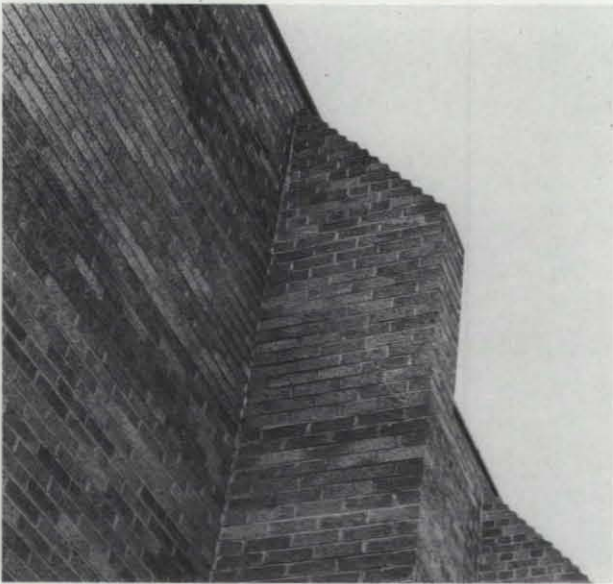
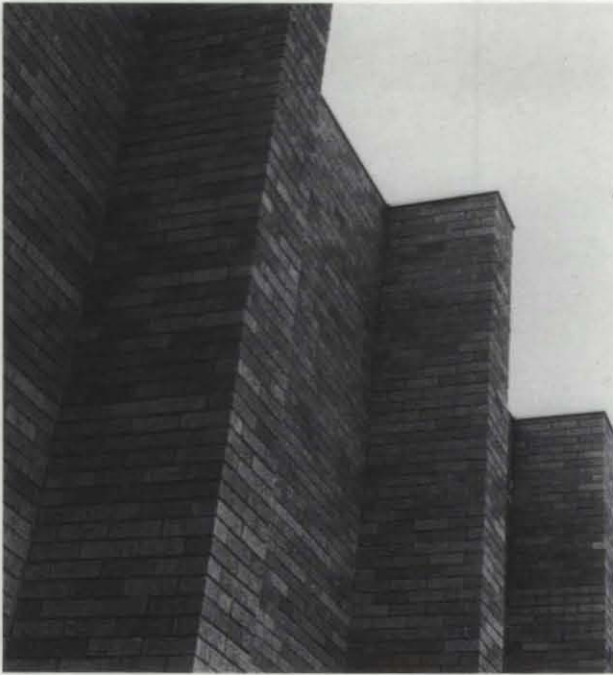
When using a variety of bricks (for example, a different facing on the fins and outer leaf from those used in the inner leaf of the cavity), care must be taken to see that the bricks are compatible, particularly with regard to thermal and moisture movement to prevent critical differential movement occurring which may damage or affect the structural behaviour of the wall. The design calculations too, under such

Design of brick fin walls in tall single-storey buildings

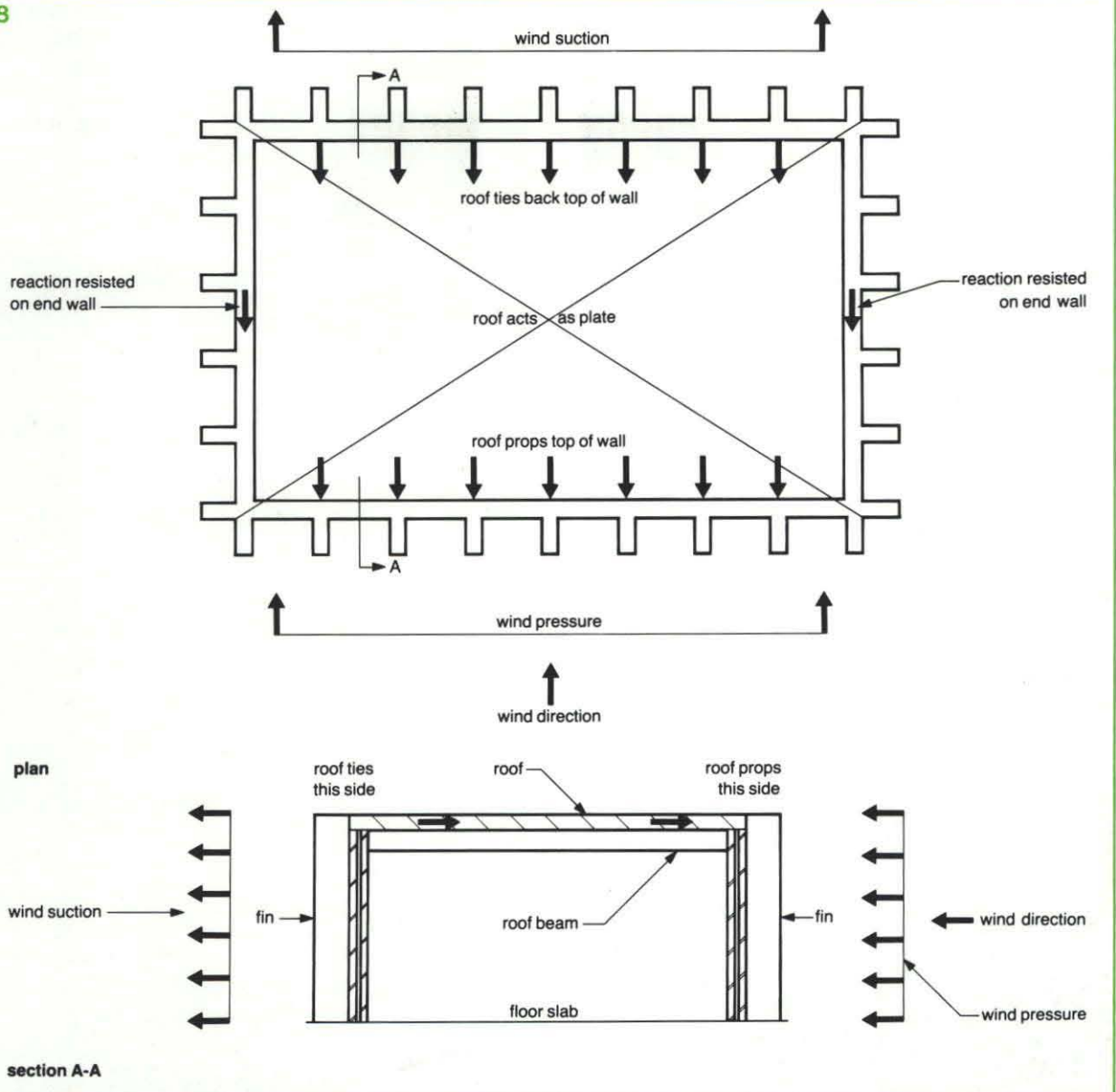


Above Two examples of fins stopped off below the roof line.

*Below, left and right Fins flush with the roof line. Note access to downpipe located inside the fin.
 Centre left Stepped fins stopped off below the roof line.
 Centre right Raked fins stopped off below the roof line.
 Foot of page, left Raked fins carried above the roof line.
 Foot of page, right Roof brought to outside of fins.*



8

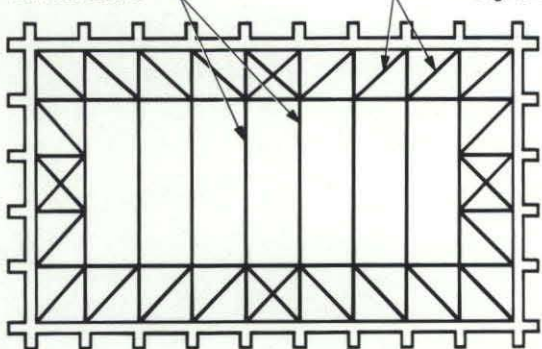


9

plan on roof girders

main roof beams

horizontal bracing to girders



circumstances, must take into account any difference in the strength of the bricks.

Where the architectural design requires that the tops of walls are not protected from falling rain, particular care must be given to the choice of a suitable brick and mortar. CP 121⁽⁴⁾ and BDA literature give guidance on the selection of suitable bricks and mortar. To ensure durability, where ordinary quality bricks are to be used in exposed situations, the manufacturers' advice should be sought.

Design of brick fin walls in tall single-storey buildings

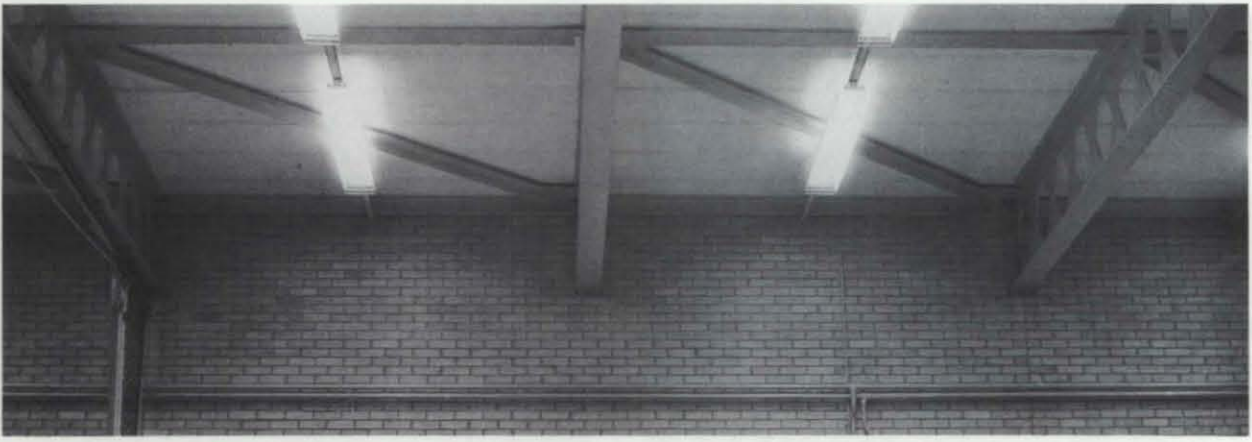
Roof

Careful consideration of the structural behaviour of the roof of the building is most important for the full economy to be achieved in the overall building cost. As with the diaphragm, to obtain greatest economy, the roof of the fin wall building should be used as a horizontal plate to prop and tie the top of the walls, and to transfer the resulting horizontal reactions to the gables or other transverse walls of the building (see Fig 8). Assuming such transverse walls to be spaced within a reasonable span for the roof plate.

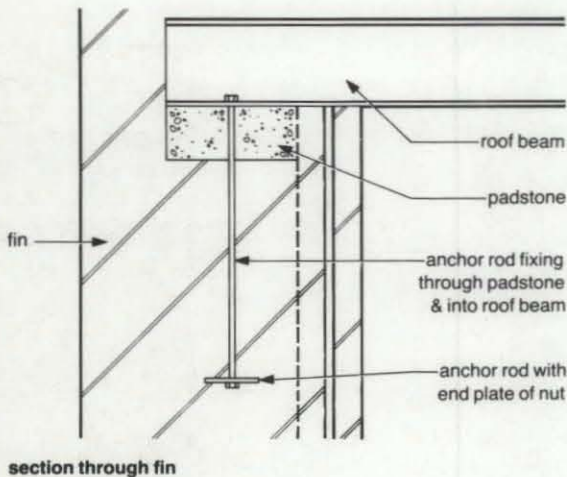
In Fig 8, the plan and section indicate the structural action of the roof plate in resisting and transferring wind forces.

In some cases the roof decking (sometimes acting in conjunction with a concrete ring beam) is all that is required to give the necessary plate action but, in other cases, horizontal roof bracing is necessary (see Fig 9).

In cases where only the ring beam and decking form the plate, it is necessary to make sure that



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Above Fin wall construction offers a wide choice of roof structures, materials and methods. In conjunction with a suitable capping beam, the roof is used as a horizontal plate member to prop and tie the tops of the walls, and to transfer the horizontal reactions due to wind to the gable or transverse walls.

from the tops of the walls on which the wind is acting to the transverse walls of the building.

If no capping beam is used, and the roof dead load is small, the main roof beam often requires to be strapped down and this can be done using rods from the padstone taken down to a suitable level to ensure sufficient dead load to resist uplift (see Fig 10).

The roof decking can be constructed from a variety of materials and supported in many ways. The final choice depending on the criteria related to the particular properties required for the building and the economy of construction.

When considering the cost of the decking, it is

the deck and its fixing are stiff enough and strong enough to control the movement and forces involved. When a concrete ring beam is used, it should be designed to help transfer the wind forces

necessary to take account of its ability to act as a roof plate. All necessary bracing should be included to give total roof costs, in order that the overall economy of the building can be examined, rather than individual items.

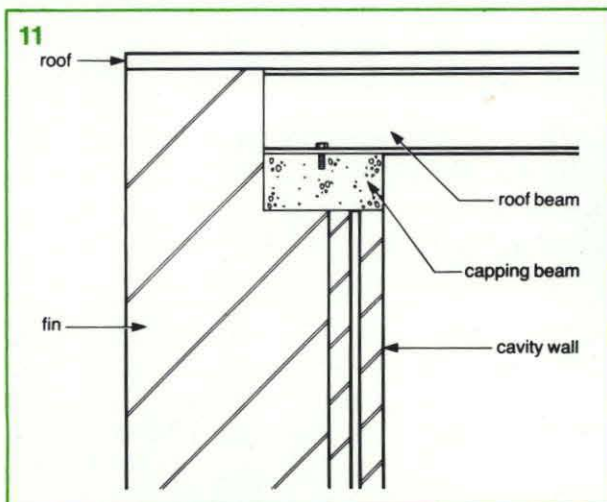
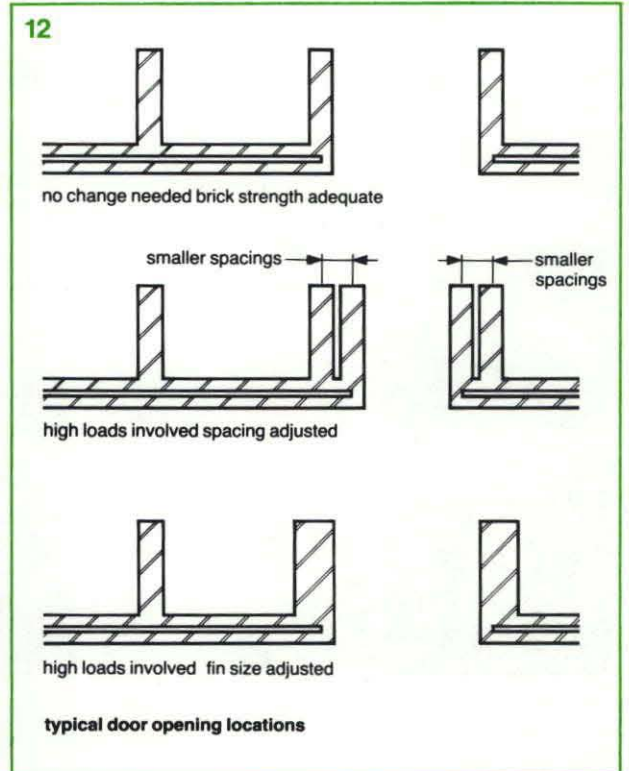
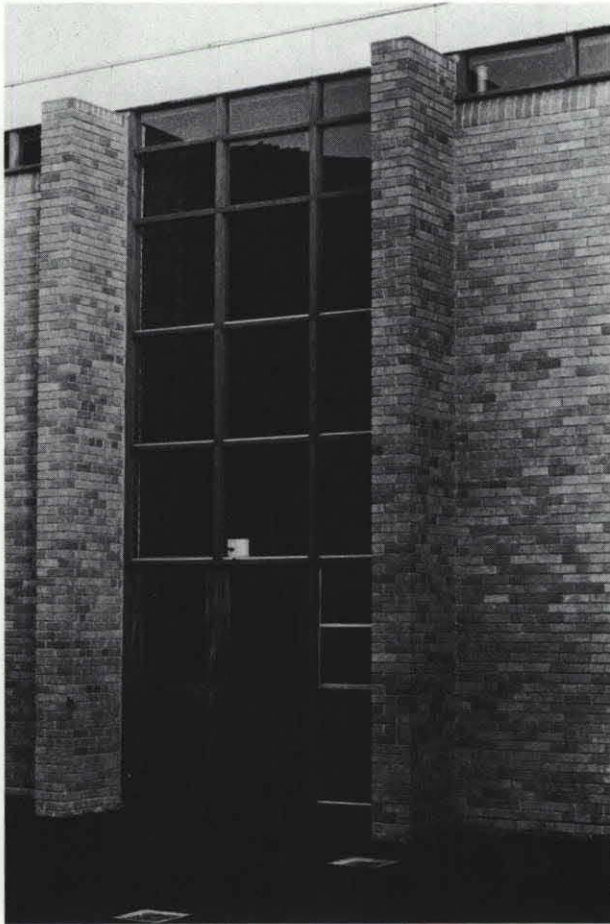
Care must be taken to see that all connections transferring wind forces, ie, deck to beam, beam to wall, deck to wall, beam to brace, wall to brace, etc are adequately designed for the forces involved. In cases where large uplift forces from the roof require a substantial anchorage, a concrete capping beam might be provided to which the main roof beams can be fixed. The capping beam can be used as a main boom for the horizontal bracing which may be required for the roof's plate action.

In cases where a concrete capping beam is to be

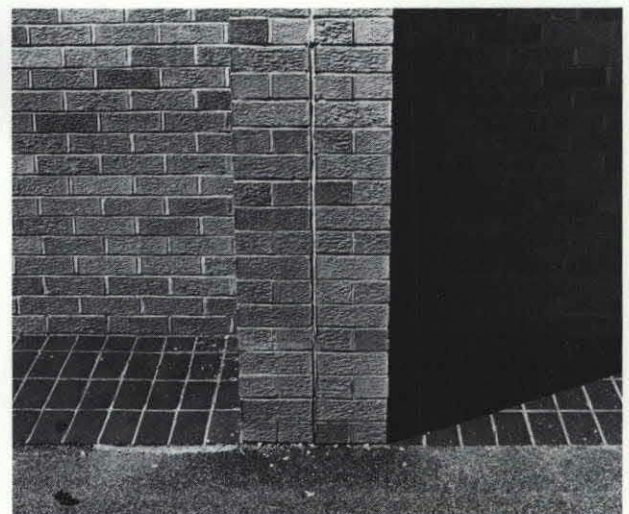
used, it is usually better constructed in precast sections of full bay lengths jointed with a joint detail capable of transferring the forces in this location. The use of precast concrete capping beams avoids the problems relating to supporting shuttering at a high level and preventing grout runs over facing brickwork, particularly where the beam is wider than the cavity wall (see Fig 11)

Openings in walls

Large openings required in the main walls can sometimes create high local loading conditions from wind and vertical loads, particularly around beam bearings. In such locations a beam or lintel is normally used to span over the opening, and an adjustment to either the fin spacing or fin cross section can usually be made to carry the increased loads involved (see Fig 12).



Design of brick fin walls in tall single-storey buildings



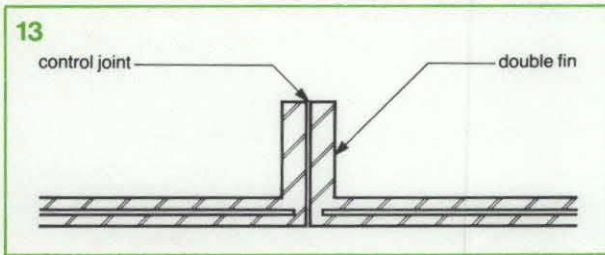
Left Floor to ceiling opening, St Mary's College, Crosby. Architects: Weightman & Bullen. Structural engineers: W. G. Curtin & Partners.

Above Double fin movement joint.

Joints

Movement joints for shrinkage and expansion are required at the appropriate centres related to the type of bricks and mortar being used, the differential temperatures expected, and in accordance with the current recommendations for brickwork in CP 121.

The joints can easily be accommodated by a double fin, one on each side of the joint (see Fig 13). It is important to consider carefully the type of joint filler to be used, and to provide adequate supervision during construction to ensure that there are no restrictions to movement.



Thermal insulation

Most buildings constructed to date on the fin wall principle have not been subject to statutory requirements for thermal insulation. Since June 1979, however, similar buildings are required to be in accordance with Part FF of the Building Regulations. The necessary insulation will be the same as any other cavity wall, and can easily be achieved by filling the cavity or partially filling the cavity with a suitable insulant. The final choice of insulant will depend on the situation and the design of the building. See BDA 'Energy Conservation, Thermal Insulation of Brick Buildings complying with Part FF of the Building Regulations'⁽⁵⁾, and BDA Design Note No 2⁽⁶⁾.

Sound insulation

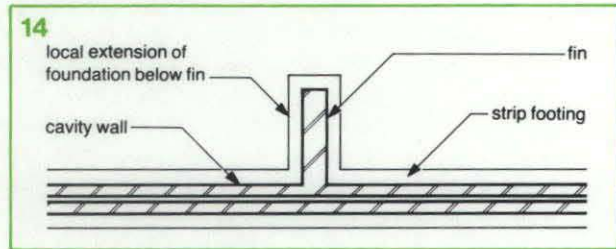
There are no statutory requirements, at present, for sound insulation in the types of building where fin wall construction has been or is likely to be used. However, the fin wall itself will substantially reduce the transmission of noise from within or without the building and will perform at least as well as a brickwork party wall in housing.

Damp proof courses

In addition to the normal impervious requirements for the horizontal dpc, it is important to select a material which has the necessary resistance to sliding and squeezing out under horizontal and vertical loadings respectively. Should it be considered desirable to transfer flexural tension to the foundation, or should prestressing techniques be employed, consideration should be given to the use of an engineering brick or slate dpc to BS 743⁽⁷⁾. It should be noted, however, that it is not recommended that the flexural tensile resistance at dpc level be exploited in the case of the propped fin unless a much more detailed analysis of the deflection of the prop is considered (see Section 2, Design bending moments).

Foundations

In most normal ground conditions, the foundations can consist of strip footings under the main perimeter walls with local extension under the fin locations (see Fig 14).



This type of footing is usually adequate for most locations but, of course, the foundations for each individual building must be determined from consideration of the particular site and ground conditions.

Temporary propping

Like most other walls, the fin wall is in a critical state during erection and prior to the roof being constructed and fixed. During this period, therefore, the contractor must take the normal temporary precautions such as propping the walls with the bricklayers' scaffolding or other means to ensure that the walls remain undamaged.

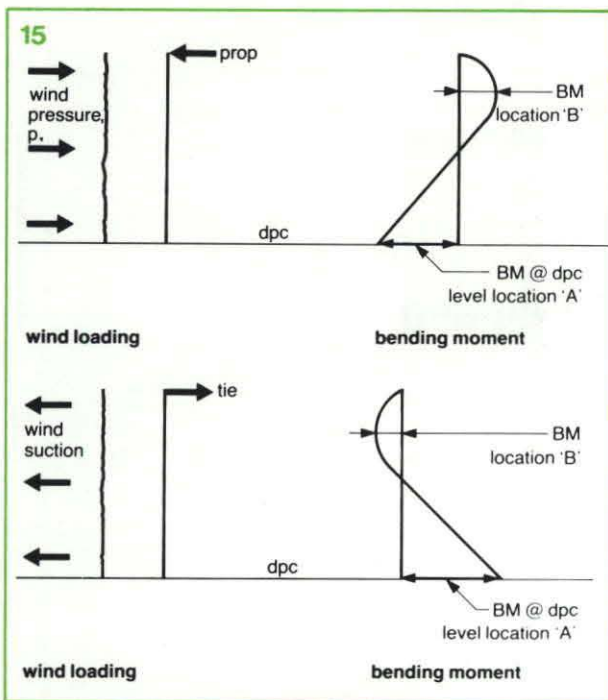
Structural design method

The main calculations involved in the design of fin walls are for the critical conditions of combined dead and wind loading. These take into account the maximum uplift and maximum bending flexural stresses.

The flexural compressive stresses involved when combined dead, superimposed, and wind loading are applied can be critical, particularly when the fin is bending about its weaker axis and the stresses at the extreme end of the fin are considered. The choice of brick and the mortar must, therefore, take into account the tensile and compressive strength required, and the durability needed for the individual building. The walls are assumed to act as 'propped' cantilevers, with the roof acting as the prop and transferring the propping forces to the transverse walls (see Fig 8), and 'fixed' at the base by virtue of their self-weight. The plate action of the roof will allow some small movement at the prop location, and the stiffness of the wall will vary due to the effects of the gravitational loads and the loss of flexural tensile resistance at dpc level.

Within the height of the wall, there are two locations of critical bending moments, these occur at dpc level, location A, and part way between dpc and roof level at location B (see Fig 15). Due to the unsymmetrical nature of the fin, it is important to consider both directions of wind loading in order to determine the critical stress.

The calculations are carried out on a trial and error basis by adopting a trial section and then



checking the stress conditions. For detailed discussion and a design example see Sections 2 and 3.

EXPERIENCE AND PERFORMANCE OF FIN WALLS

At least a dozen fin wall buildings have been completed and more are under construction or in the design stage. The buildings already completed are, on the whole, situated on exposed sites in the North West of England.

Those constructed before 1976 have already survived:

- (a) the worst gales on record in January 1976
- (b) the hottest summer on record
- (c) the worst drought
- (d) the wettest autumn

These, and the more recent examples, have also survived one of the most severe winters this century.

The buildings have all performed successfully, and no problems have developed as a result of the method of construction. In particular they have, both internally and externally, withstood the hard usage associated with sports halls without requiring maintenance.

Further applications

In addition to the use of fin walls for new buildings, they have also been found very useful for strengthening existing buildings. In one particular case the rear wall of a grandstand, which was showing signs of becoming unstable, was strengthened by bonding into it, at predetermined centres, a series of brick fins designed to resist the excessive loading likely to be applied.

A further application was the use of post-tensioned fins to strengthen a retaining wall to an existing basement, where a change of use resulted in increased lateral loading which made it bulge and crack and become unstable. The post-tensioned brick fins proved easy to construct and economical when compared with alternative forms of construction.

Future progress

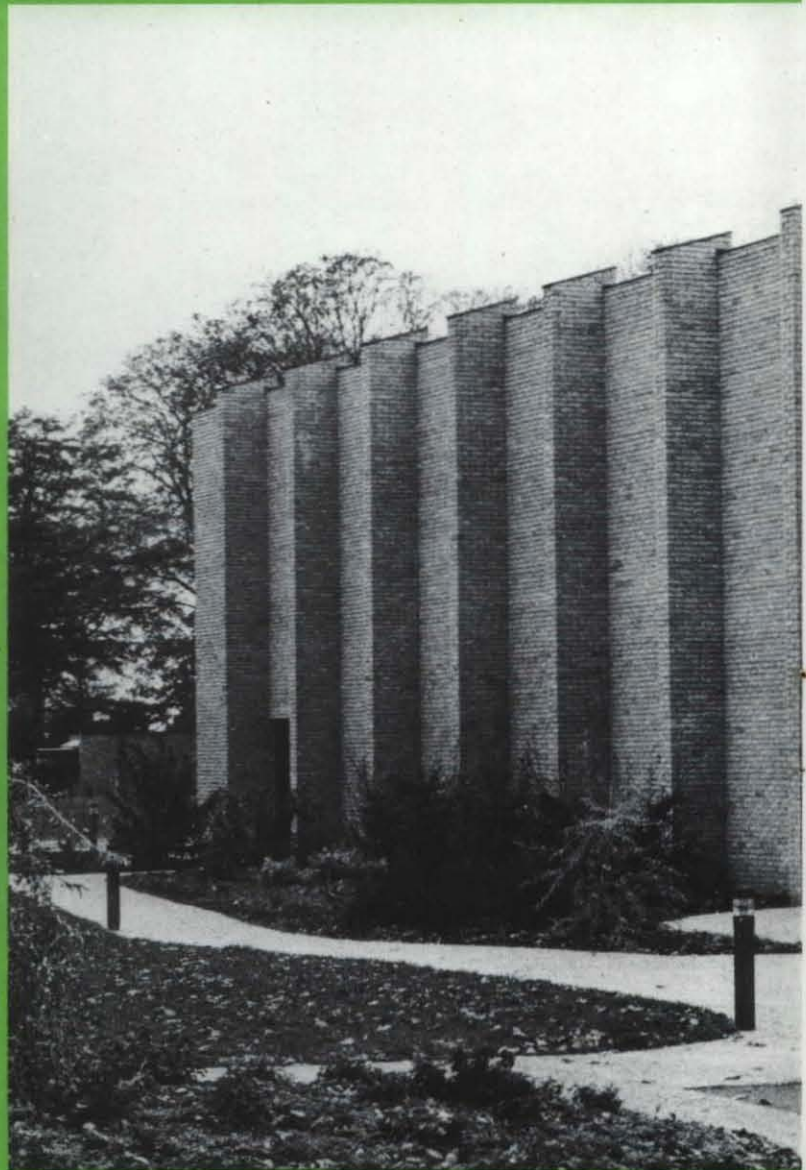
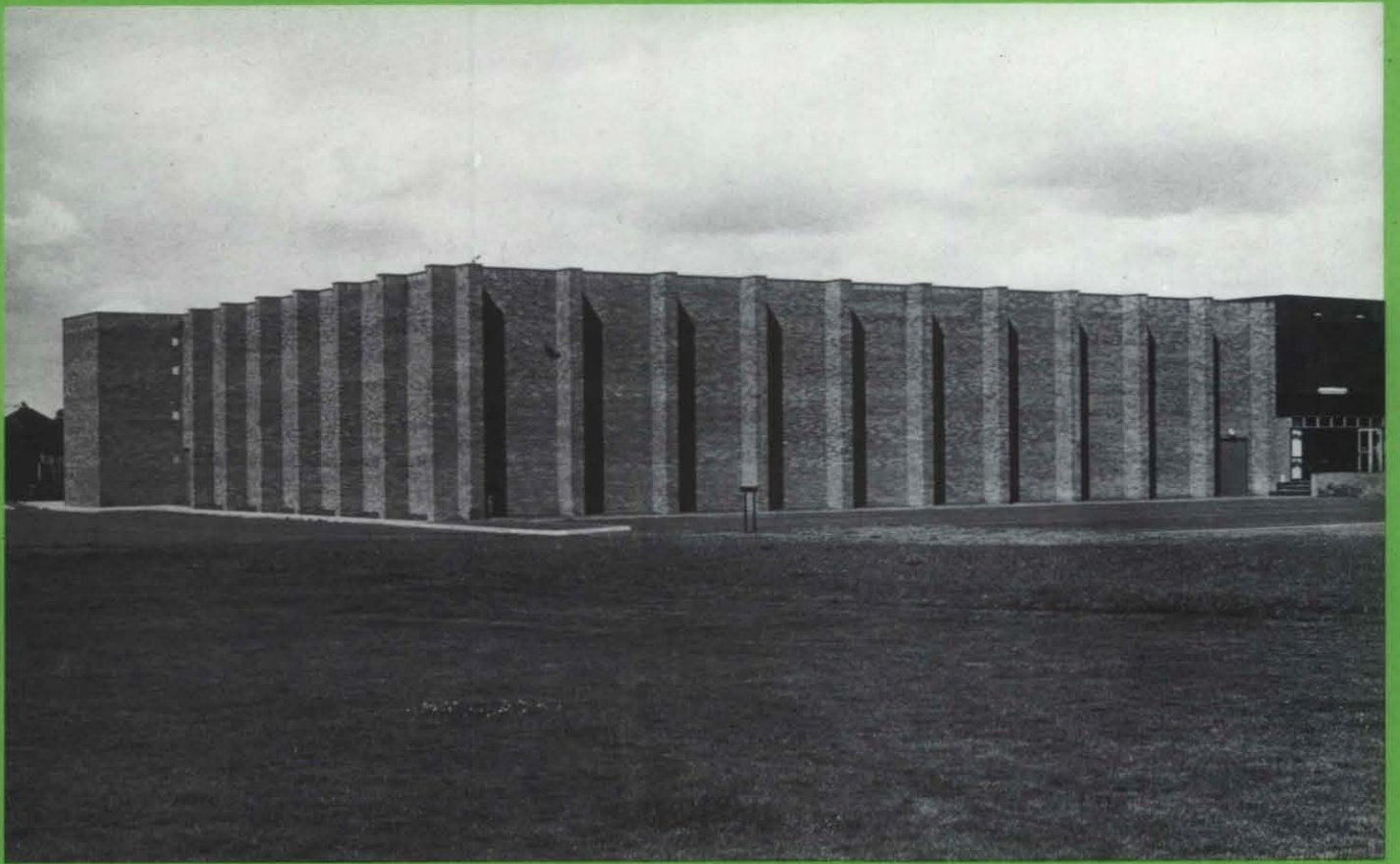
Although the fin wall was developed mainly for use in tall, single-storey wide-span buildings, it has become apparent to the authors that it has a much wider application. For example, the post-tensioned fin wall used as a retaining wall is attractive both economically and visually and has great potential for the future.

The use of fin walls in conjunction with spine walls can result in multi-storey buildings of unrestricted floor areas for office buildings, hospital ward blocks and other building forms which cannot tolerate the restrictions of crosswall or cellular construction.

Below and top left overleaf Rudheath County Secondary School. Architects: Wilson & Womersley. Structural engineers: W. G. Curtin & Partners.



Design of brick fin walls in tall single-storey buildings



Centre The Grange Sports Hall, Ellesmere Port. Architects: Paterson, Macaulay & Owens. Structural engineers: W. G. Curtin & Partners.
Above Chester Catholic High School. Architects: Ellis Williams Partnership. Structural engineers: W. G. Curtin & Partners.
Top right and right County Officers Sports Club, Upton, Chester. Architects: Cheshire County Architects Department. Structural engineers: W. G. Curtin & Partners.



Design of brick fin walls in tall single-storey buildings

CRITICAL DESIGN CONDITION

The calculations which follow are based on reasonable assumptions, some of which are as yet unsupported by research. However, structures which have been designed in accordance with these theories and assumptions have performed, and are performing, successfully.

For tall single-storey buildings, the critical design condition is rarely governed by axial compressive stresses but by the wall's resistance to lateral forces from wind pressures. The flexural tensile stresses generally govern the design and it is, therefore, beneficial to either reduce the flexural tensile stresses by reducing the maximum applied bending moment, or by increasing the section modulus and/or increase the compressive stresses.

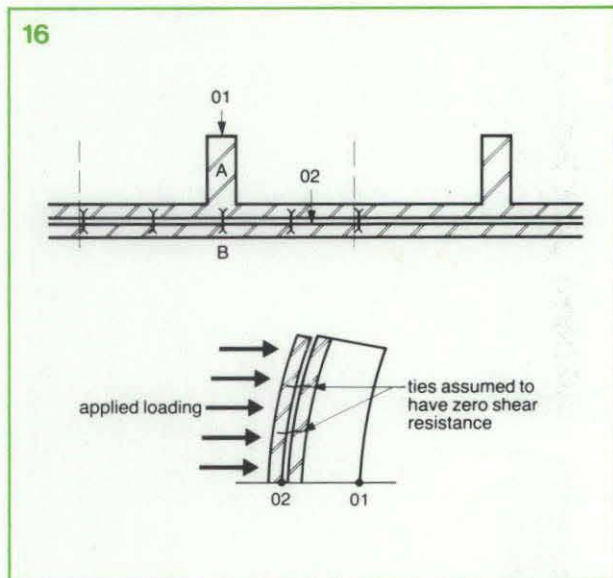
This can be achieved by:

- (a) using the roof as a plate (see Fig 8) to prop the wall, thus reducing the bending moment when compared with a free standing cantilever, and;
- (b) using a T section – fin wall;
- (c) using post-tensioning to increase the compressive stresses and to decrease section sizes.

The design of post-tensioned fin walls will be included in a future publication and is not covered in this design guide.

Interaction between leaves

As shown in Fig 16, the fins are bonded to one of the leaves of a cavity wall and considered as a T section combining the bonded leaf with the fin. The other leaf is considered as a secondary member, the cavity ties being assumed to be unable to transmit significant vertical shear forces but capable of transmitting horizontal forces across the cavity width. The type of tie assumed for this condition is the galvanised vertical twist tie to BS 1243⁽⁹⁾, and under most conditions this is adequate. However, the designer should satisfy himself that the ties are suitable for the exposure conditions in which they are employed, and that they can transfer the design forces adequately.



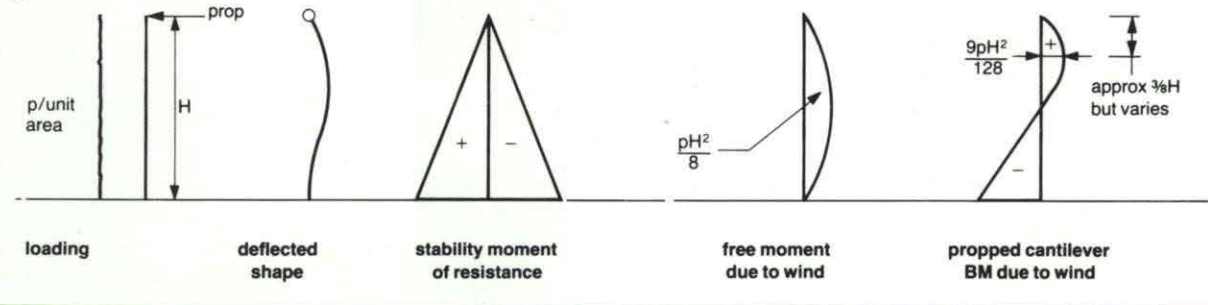
Effective section

Because of the unsymmetrical shape of the member, the geometrical properties of the effective sections, when combined bending and direct forces are considered, can vary greatly under changes in loading particularly if a 'cracked section' is being analysed. It is, therefore, important when considering the stability moment of resistance to also consider carefully the effective section being stressed and the effects of any cracked portion on the general performance of the wall. The flexural stresses must be kept within those recommended in the Code of Practice but, at dpc level, the majority of damp proof courses must be considered to have no resistance to flexural stress, and at this level a 'cracked section' is often assumed. The moment of resistance at this level becomes the gravitational moment of resistance for the worst loading combination, which is generally that of dead plus wind loading.

DESIGN CONSIDERATIONS

The various loading combinations and their effect on the stress conditions must be carefully considered, therefore, one of the first calculations is that of assessing the loads:

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- (1) calculate the positive and negative wind pressure
- (2) calculate the dead, imposed and wind loading on the roof and walls.

Having obtained the loading condition, it is important before progressing with the design to make sure that the assumed behaviour of the structure is understood. In the case of the fin wall being used on tall single-storey buildings, it is assumed that the wall acts as a propped cantilever (see Fig 8 & Fig 17) where the 'fixed end moment' is that due to the vertical loads, and is known as the stability moment.

Uplift

It is most important to take account of roof uplift forces when considering the worst design condition. It is also important to note that the critical section at or near the base of the wall is usually at the location of the dpc where little or no tension is permissible, depending on the chosen membrane.

Determine maximum critical stresses

It is necessary in the calculations to determine the maximum critical forces, moments and stresses in the wall, which, for a normal propped cantilever, occur at or near the base of the wall and at a point approximately $\frac{3}{8}H$ from the top of the wall. However, the propped fin wall will vary from this as explained below.

Differential stiffness within height of wall

For a uniformly distributed load on a propped cantilever of constant stiffness with a rigid prop, the bending moment would be as shown in Fig 18.

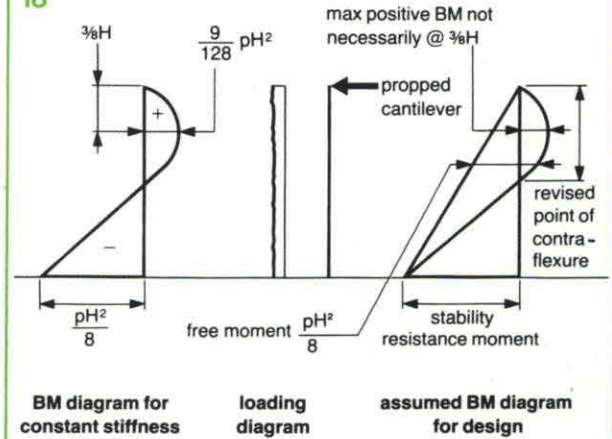
However, for the brick fin wall shown, some deflection will occur at the prop location, and the wall strength will vary within the height of the wall due to the variation at each level in the axial load.

It would, therefore, be merely coincidence if the stability moment at the base was exactly equal to $\frac{pH^2}{8}$ which is the condition for the straightforward propped cantilever.

Design bending moments

As the applied bending moment is increased, the stability moment at the base for the same axial

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load will not decrease but slight cracking and rotation of the base of the wall will occur and produce increased bending at the upper location. It is, therefore, more realistic in the design to first calculate the stability moment at the base dpc level taking account of the appropriate partial safety factor for loads

The design load free bending moment can then be superimposed upon the stability moment diagram (see Fig 17). The position and magnitude of the maximum positive bending can then be determined and these stress conditions checked.

Check both directions of bending

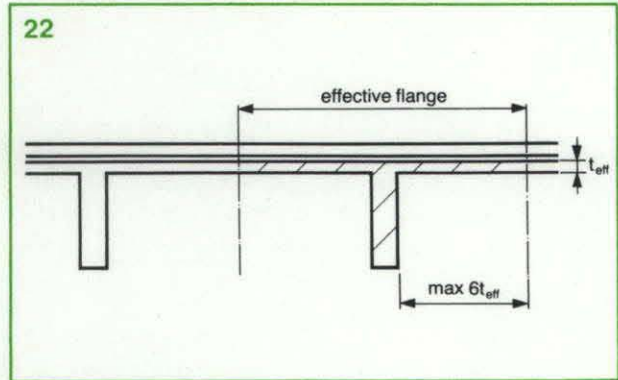
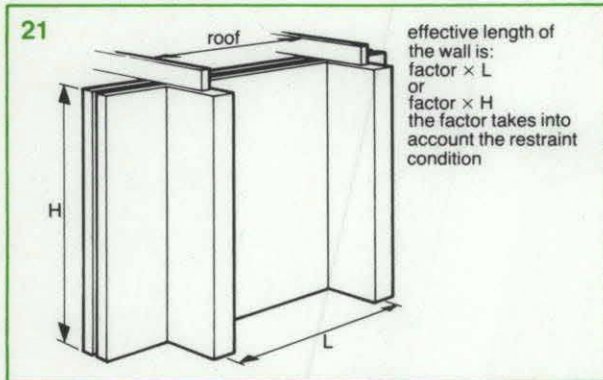
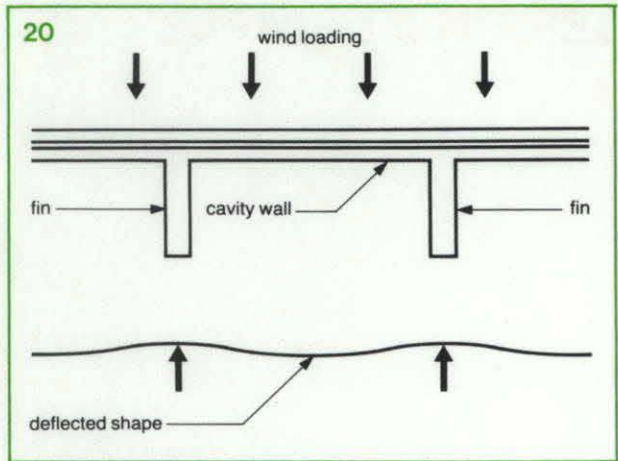
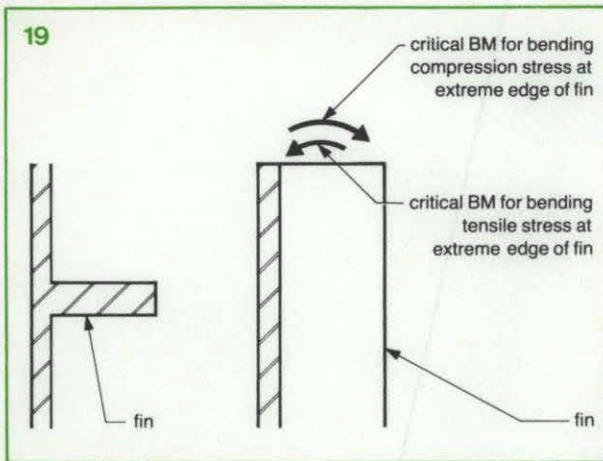
It is important, when considering the bending moments on the fin, to check for the bending moments in each direction at each level, since the critical stress conditions will not necessarily result from the same direction of applied bending moment (see Fig 19).

TRIAL AND ERROR DESIGN

It is apparent from the suggested procedure that the design must commence on a trial and error basis, first choosing a reasonable section and then checking the stress conditions which exist.

Spacing of fins

The choice of a suitable section must take into account the cavity wall's ability to act suitably with the fin to both transfer wind forces to the overall section and to prevent buckling of the flange of the T section. This involves choosing a suitable spacing for the fin to control both these conditions and to take into account economic



spacing of the roof beams. The spacing of the fin is, therefore, governed by the following conditions:

(a) The cavity wall acting as a continuous horizontal slab subjected to wind load, spanning between the fins (see Fig 20).

(b) The cavity wall's ability to support vertical load without buckling. This is governed by the slenderness ratio of the wall, BS 5628, clause 28, (see Fig 21).

(c) The ability of the cross section to resist the applied loading with the leaf and fin acting together to form a T beam.

The effective flange of the T beam (see Fig 22) is limited to the least of:

- (i) the distance between the centres of the fins
- (ii) the breadth of the fin plus twelve times the effective thickness of the bonded leaf
- (iii) one-third of the effective span of the fin.

It should be noted that clause 36.4.3 of BS5628 embraces two of these conditions with reference to pierced walls but, since it is felt that the distribution of stress into the flange is also related to the *span* of the fin (in a similar manner to a reinforced concrete T beam), a span related limit is also necessary.

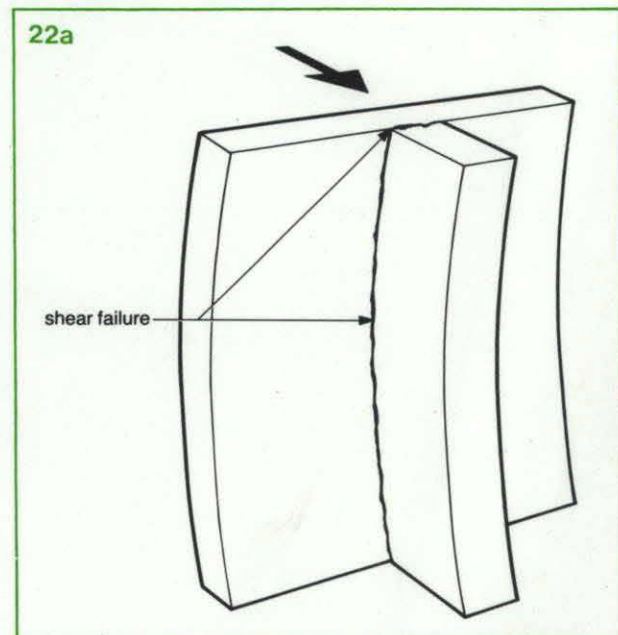
(d) The vertical shear forces between the fin and the bonded leaf resulting from the applied bending moment on the T section (see Fig 22a).

(e) The economic spacing of the main roof supports.

It should be noted that whilst item (c) restricts the flange length for the design of the fin, the actual distance between the fins can be greater.

Typical fin sizes are 1–2m deep at spacings of 3 to 5 m and 1½ bricks (327 mm) or 2 bricks (440 mm) wide. Some typical sections and their properties are shown in Table 1. The length and thickness of the fin is governed by the tendency of the outer edge to buckle under compressive bending stress.

The roof plate action and the stresses in the transverse walls which provide the reactions to the plate must be checked.

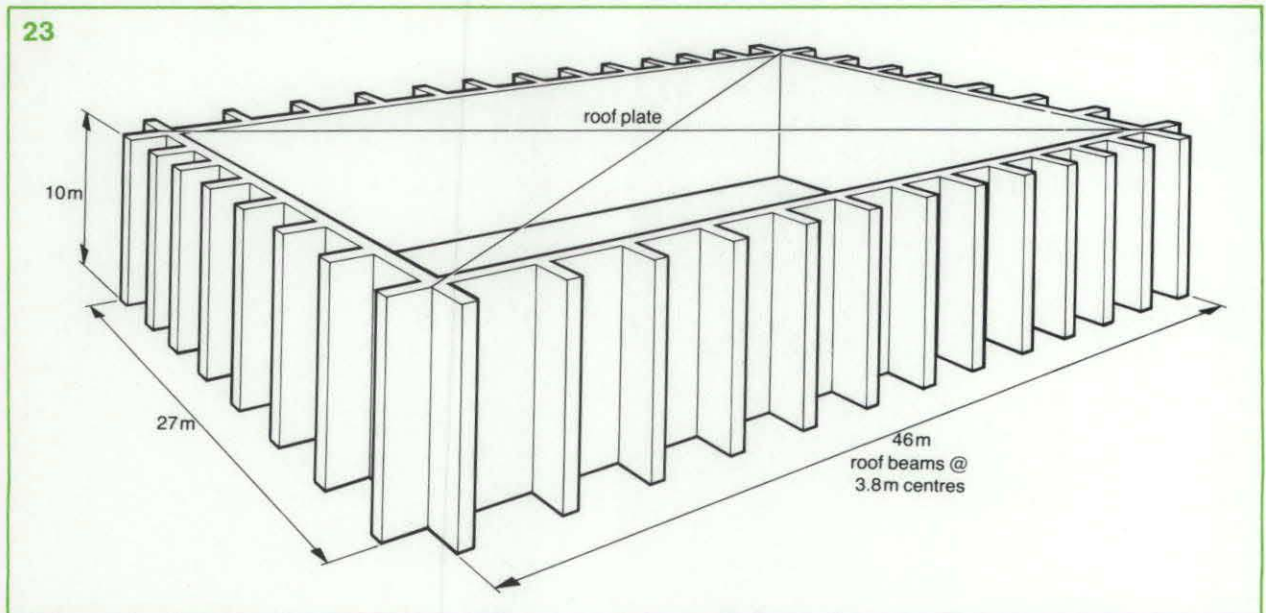


SYMBOLS

Where applicable, the symbols used in BS 5628 have been adopted. However, because BS 5628 does not cover the design of fin walls, a number of additional symbols are required and, therefore, a full list of the symbols used in this guide is provided below.

c_{pe}	External pressure coefficient
c_{pi}	Internal pressure coefficient
e_x	Eccentricity of axial loading
f_k	Characteristic compressive strength of masonry
f_{kx}	Characteristic flexural strength of masonry (tensile)
f_{ubc}	Flexural compressive stress at design load
f_{ubt}	Flexural tensile stress at design load
G_k	Characteristic dead load
I_{NA}	Second moment of area about neutral axis
H	Height of fin wall
H_{eff}	Effective height
L	Spacing of fins, centre to centre
l_a	Lever arm
MB	Base moment
MR_s	Stability moment of resistance
M_w	Wall moment
p	Basic wind pressure
p_{ubc}	Design flexural compressive stress = $\frac{\beta f_k}{\gamma_m}$
p_{ubt}	Design flexural tensile stress = $\frac{f_{kx}}{\gamma_m}$
Q_k	Characteristic superimposed load
SR	Slenderness ratio
W_1	Own weight of effective Tee profile per metre height
W_{k1}	Design wind pressure (windward wall)
W_{k2}	Design wind pressure (leeward wall)
W_{k3}	Design wind uplift (on roof)
Y_1	Dimension – neutral axis to end of fin
Y_2	Dimension – neutral axis to flange face
Z	Section modulus
Z_1	Minimum section modulus = $\frac{I_{NA}}{Y_1}$
Z_2	Maximum section modulus = $\frac{I_{NA}}{Y_2}$
β	Capacity reduction factor
γ_f	Partial safety factor for loads
γ_m	Partial safety factor for materials
Ω	Trial section coefficient = $(W_1 \times Y_2)$ per m height

DESIGN PROBLEM



A warehouse measuring 27 m × 46 m on plan, and 10 m high, is shown in Fig 23. The building is to be designed in brickwork, using fin wall construction for its main vertical structure. The fins are to project on the external face, and the wall panels between the fins are to be of 255 mm brick cavity construction. There are no internal walls within the building. The building is part of a major development where extensive testing of materials and strict supervision of workmanship will be employed.

The architect has selected particular facing bricks which are shown to have a compressive strength of 30 N/mm² and a water absorption of 10%. The facing bricks will be used both inside and outside the building.

DESIGN APPROACH

Introduction

BS 5628 offers three options for the design of laterally loaded walls:

(a) Clause 36.4.3 in which the design moment of resistance of wall panels is given as $\frac{f_{kx}Z}{\gamma_m}$

and

(b) Clause 36.8 which offers two further options:

- (i) design lateral strength equated to effective eccentricity due to lateral loads,
- or
- (ii) treating the panel as an arch.

The last option can seldom be applied to single-storey buildings, due to inadequate arch thrust resistance. The remaining two options take no account of flexural compressive stresses which, in the fin wall design concept, certainly require careful consideration.

For this reason, it has been considered necessary, to properly explain the mechanisms involved, to diverge from the BS 5628 concept of equating design loads to design strengths. The analysis considers stresses due to design loads and relates these to allowable flexural stresses in both compression and tension.

(1) Characteristic loads

(a) Wind forces

The basic wind pressure on a building is calculated from a number of variables which include:

- (i) location of building, nationally
- (ii) topography of the immediate surrounding area
- (iii) height above ground to the top of the building
- (iv) building geometry.

For the appropriate conditions, the basic pressure and local pressure intensities are given in CP 3, Chapter V, Part II ⁽⁹⁾.

In this example, these values are assumed to have been computed as:

Dynamic wind pressure, $q = 0.74 \text{ kN/m}^2$
 c_{pe} on windward face $= 0.8$
 c_{pe} on leeward face $= -0.55$
 c_{pi} on walls either $= +0.2$ or -0.3
 Gross wind uplift $= c_{pe} + c_{pi} = 0.53$

Therefore characteristic wind loads are:

Pressure on windward wall $= W_{k1} = (c_{pe} - c_{pi}) q = (0.8 + 0.3) 0.74 = 0.814 \text{ kN/m}^2$
 Suction on leeward wall $= W_{k2} = (c_{pe} - c_{pi}) q = (0.55 + 0.2) 0.74 = 0.56 \text{ kN/m}^2$
 Gross roof uplift $= W_{k3} = (c_{pe} + c_{pi}) q = 0.53 \times 0.74 = 0.39 \text{ kN/m}^2$

(b) Dead and superimposed loads

(i) Characteristic superimposed load

$= Q_k = 0.75 \text{ kN/m}^2$

(Assuming no access to roof, other than for cleaning or repair, in accordance with CP3, Chapter V, Part 1.)

(ii) Characteristic dead load $= G_k$

Assume: metal decking $= 0.18 \text{ kN/m}^2$
 felt and chippings $= 0.27 \text{ kN/m}^2$
 o.w. roof beams $= 0.15 \text{ kN/m}^2$

Total $G_k = 0.60 \text{ kN/m}^2$

(2) Design loads

The critical loading condition to be considered for such a wall is usually wind + dead only, although the loading condition of dead + superimposed + wind should be checked.

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

Design wind load $= 1.4 W_k$ or $0.015 G_k$
 whichever is the larger.

Therefore, by inspection, the most critical combination of loading will be given by:

Design dead load $= 0.9 \times 0.6 = 0.54 \text{ kN/m}^2$

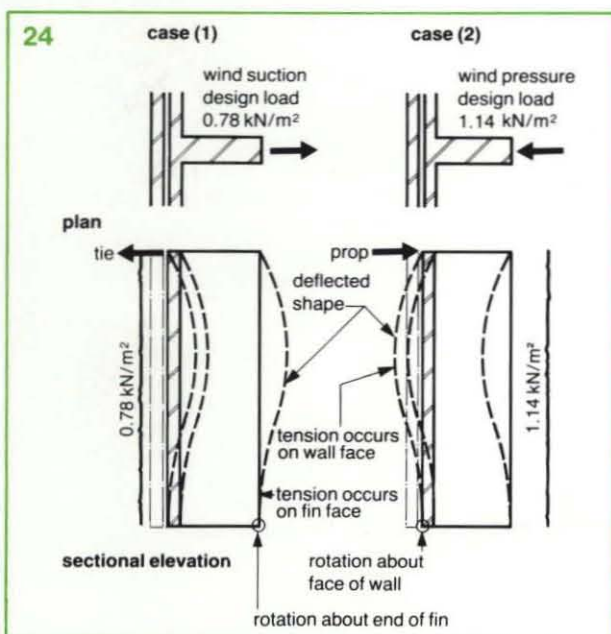
Design wind loads:

Pressure, from $W_{k1} = 1.4 \times 0.814 = 1.14 \text{ kN/m}^2$

Suction, from $W_{k2} = 1.4 \times 0.56 = 0.78 \text{ kN/m}^2$

Uplift, from $W_{k3} = 1.4 \times 0.39 = 0.54 \text{ kN/m}^2$

Design dead - uplift $= 0.54 - 0.54 = \text{zero}$



(3) Design cases

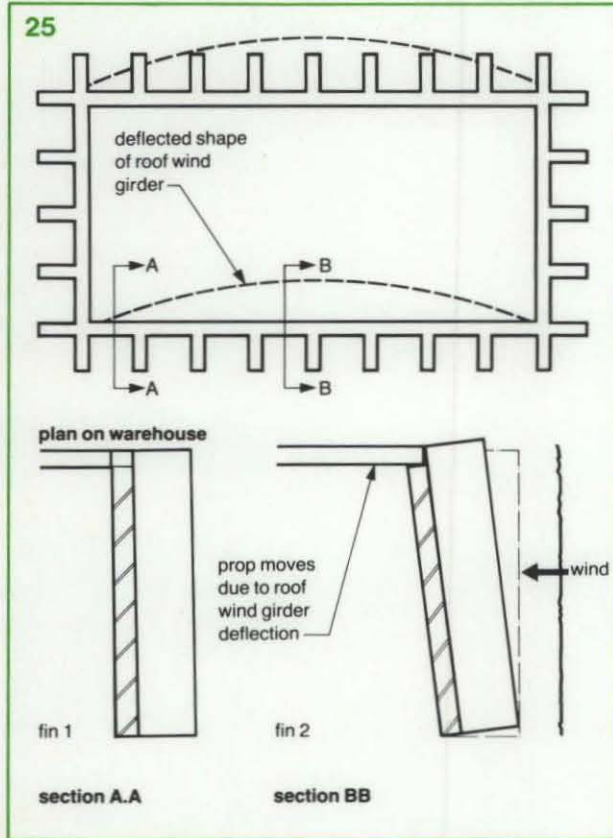
Inner leaf offers minimal resistance and is ignored in calculations apart from assisting stiffness of flange in bending.

Note Vertical loading from own weight of effective section only as uplift exactly cancels out roof dead loads.

(4) Deflection of roof wind girder

The wall is designed as a propped cantilever and utilises the fins bonded to the outer leaf to act as vertical T beams resisting the flexure.

The prop to the cantilever is provided by a wind girder within the roof decking system (the design of this wind girder is not covered by this guide). The reactions from the roof wind girder are transferred into the lateral gable shear walls at each end of the building. Horizontal deflection of the roof wind girder, reaching a maximum at midspan, has the effect of producing additional rotation at base level (see Fig 25) and this results in a less critical stress condition. However, the critical stress conditions are generally experienced in the end fins where the roof wind girder deflection is a minimum.



(5) Effective flange width for T profile

The dimensional limits for the effective length of the wall permitted to act as the flange of the T profile are given in BS 5628, Clause 36.4.3(b), as $6 \times$ thickness of wall forming the flange, measured as a projection from each face of the fin, when the flange is continuous. In this design example, as will be the general case in practice, the wall forming the flange is the outer leaf of a cavity wall, as defined in BS 5628, Clause 29.1.1. It is, therefore, reasonable to take advantage of the stiffening effect of the inner leaf in resisting buckling of the outer leaf, when acting as the flange of the T profile. The effective flange length, measured from each face of the fin, is therefore calculated as $6 \times$ effective wall thickness.

Thus:

$$\begin{aligned} \text{effective wall thickness} &= \frac{2}{3}(102.5 + 102.5) \\ &= 137 \text{ mm} \\ \text{effective flange width} &= (6 \times 137) + 327 + (6 \times 137) \\ &= 1971 \text{ mm.} \end{aligned}$$

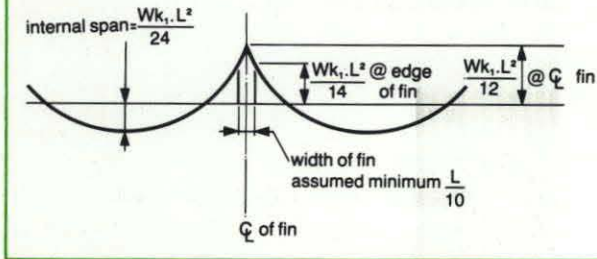
(6) Spacing of fins

The spacing of fins has been discussed on page 7 – but one aspect only. The capacity of the wall panel to span between the fins is considered here.

There is no doubt that the support provided for the wall panel at foundation level will assist in resisting the flexure due to wind forces. However, this assistance will diminish at the higher levels of the wall panel, and the wall should be designed to span purely horizontally between the fins.

The wall panels are taken as continuous spans and the maximum bending moments are shown in Fig 26.

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The maximum moment is $\frac{W_{k1} L^2}{14}$ at the edge of the fins, for an assumed fin width of $\frac{L}{10}$

$$\text{Design moment} = \frac{W_{k1} L^2}{14} = \frac{1.14 \times L^2}{14} = 0.0814 L^2$$

From BS 5628, Clause 36.4.3:

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

(i) f_{kx} : for water absorption 7% to 12% set in a designation (iii) mortar = 1.10 N/mm²

(ii) Z : for 2 leaves = $\frac{2 \times 0.1025^2 \times 1.0}{6} = 0.0035 \text{ m}^3$

(iii) γ_m : from BS 5628, table 4, special categories of manufacturing and construction control are applicable = 2.5

Therefore, design moment of resistance = $\frac{1.10 \times 0.0035 \times 10^9}{2.5 \times 10^6} = 1.54 \text{ kNm}$

From this check maximum span of wall panel.

Design moment = design moment of resistance
 $0.0814 L^2 = 1.54$

$$L = \sqrt{\frac{1.54}{0.0814}}$$

$$L = 4.35 \text{ m} = \text{maximum fin spacing}$$

Therefore, 3.80 m fin spacing is acceptable.

(7) Trial section

A trial section can be reasonably obtained by providing a section which has a stability moment of resistance MR_s , at the level of MB, equal to $\frac{W_{k1} LH^2}{8}$ under wind pressure loading W_{k1} i.e. when rotation at the base of the wall is about the face of the flange. For the purpose of the trial section assessment, the stability moment of resistance can be simplified to ΩH in which:

Ω = trial section coefficient from Table 1

H = height of fin wall

Therefore: $\frac{W_{k1} LH^2}{8} = \Omega H$

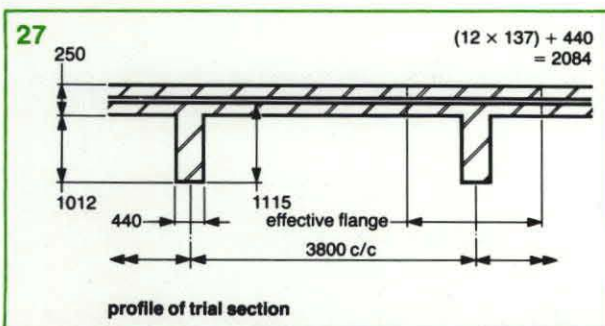
$$\frac{1.14 \times 3.8 \times 10^2}{8} = \Omega 10$$

Therefore $\Omega = 5.415 \text{ kNm/m}$ height of wall

From Table 1, select fin wall profile 'K'.

Note It is important that this trial section coefficient is used only for the selection of the trial section. A thorough structural analysis must always be carried out

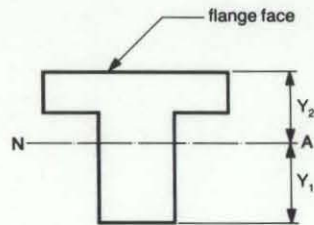
27



Fin reference letter	A	B	C	D	E	F	G	H	J	K	L	M	N	P	Q	R
Fin size (mm)	665 × 327	665 × 440	788 × 327	788 × 440	890 × 327	890 × 440	1003 × 327	1003 × 440	1115 × 327	1115 × 440	1227 × 327	1227 × 440	1339 × 327	1339 × 440	1451 × 327	1451 × 440
Effective width of flange (m)	1.971	2.084	1.971	2.084	1.971	2.084	1.971	2.084	1.971	2.084	1.971	2.084	1.971	2.084	1.971	2.084
Neutral axis Y_1 (m)	0.455	0.435	0.524	0.500	0.589	0.563	0.654	0.626	0.718	0.687	0.780	0.747	0.841	0.807	0.902	0.866
Neutral axis Y_2 (m)	0.210	0.230	0.254	0.278	0.301	0.327	0.349	0.377	0.397	0.428	0.447	0.480	0.498	0.532	0.549	0.585
Effective area (m ²)	0.386	0.4611	0.4262	0.5152	0.4595	0.5601	0.4965	0.6098	0.5331	0.6591	0.5697	0.7084	0.6064	0.7577	0.6430	0.807
o.w. of effective area per m height W (kN)	7.720	9.222	8.458	10.216	9.190	11.202	9.930	12.196	10.662	13.182	11.394	14.168	12.128	15.154	12.860	16.140
I_{NA} (m ⁴)	0.01567	0.01939	0.02454	0.0303	0.0359	0.04426	0.05021	0.06187	0.06746	0.08312	0.088	0.10848	0.11208	0.13826	0.13992	0.17277
Z_1 (m ³)	0.03441	0.0445	0.04684	0.06059	0.06096	0.07862	0.07677	0.09883	0.09395	0.12099	0.11282	0.14522	0.13327	0.17132	0.15513	0.1995
Z_2 (m ³)	0.07462	0.0843	0.09663	0.10898	0.11928	0.13536	0.14387	0.16410	0.16992	0.19421	0.19687	0.226	0.22506	0.26039	0.25487	0.2953
Trial section coefficient Ω (kNm/m)	1.6212	2.1210	2.1483	2.840	2.7662	3.6631	3.4656	4.5978	4.2328	5.6419	5.0931	6.8006	6.0397	8.0619	7.0601	9.4419

$$Z_1 = \frac{I_{NA}}{Y_1} \quad Z_2 = \frac{I_{NA}}{Y_2}$$

Trial section coefficient $\Omega = W Y_2$



(8) Consider propped cantilever action

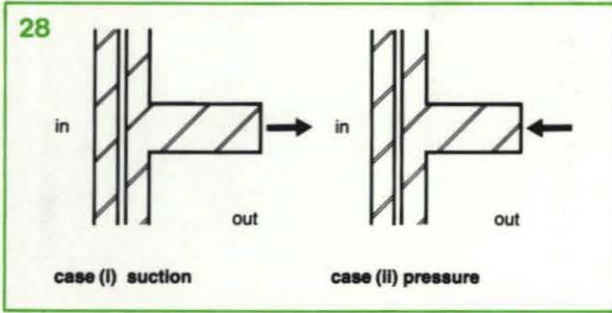
With 3.80 m fin centres, design wind loads on fins are:

Case (i), suction (see Fig 28)

$$W_{k2}L = 0.78 \times 3.8 = 2.964 \text{ kN/m of height}$$

Case (ii), pressure (see Fig 28)

$$W_{k1}L = 1.14 \times 3.8 = 4.332 \text{ kN/m of height}$$



Assuming MR_s is greater than $\frac{W_{k1} LH^2}{8}$ and zero deflection of the roof prop, the following BM diagrams can be drawn:

Case (i)

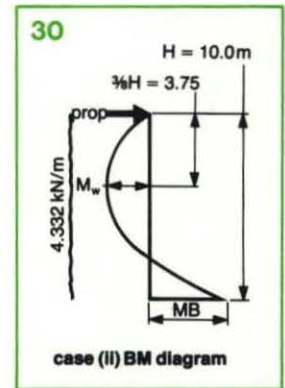
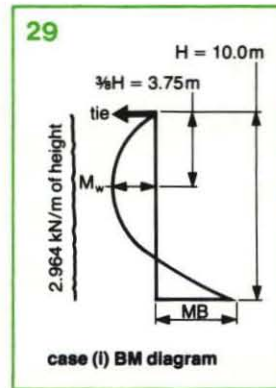
$$\text{Wall moment, } M_w = \frac{9W_{k2} LH^2}{128} = \frac{9 \times 2.964 \times 10^2}{128} = 20.84 \text{ kNm}$$

$$\text{Base moment, } MB = \frac{W_{k2} LH^2}{8} = \frac{2.964 \times 10^2}{8} = 37.05 \text{ kNm}$$

Case (ii)

$$\text{Wall moment, } M_w = \frac{9W_{k1} LH^2}{128} = \frac{9 \times 4.332 \times 10^2}{128} = 30.46 \text{ kNm}$$

$$\text{Base moment, } MB = \frac{W_{k1} LH^2}{8} = \frac{4.332 \times 10^2}{8} = 54.15 \text{ kNm}$$

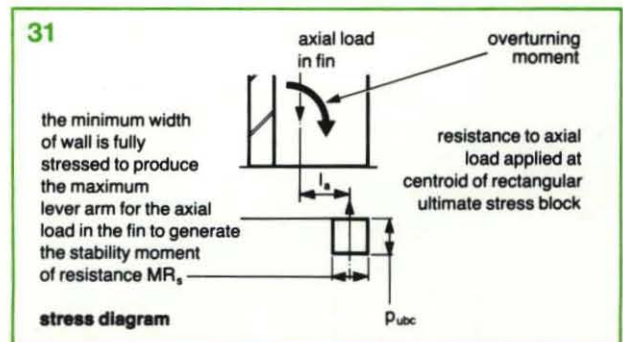


The bending moment diagrams shown in Figs 29 and 30 are applicable only if it can be shown that the stability moment of resistance of the 'cracked section' MR_s at dpc level exceeds $\frac{W_k LH^2}{8}$. This should be the first check to be carried out, and if MR_s is less than $\frac{W_k LH^2}{8}$ the base moment is limited to MR_s and the BM diagram must be redrawn plotting the free moment diagram onto the fixed end moment diagram which is produced by MR_s (see Fig 38).

(9) Stability moment of resistance

Invariably, as is the case with this design example, there will be a damp proof course at or near to the base of the wall. Few dpcs are capable of transmitting much flexural tensile stress across the bed joint, and in this example the analysis considers the 'cracked section'.

Appendix B of BS 5628 discusses the application of a rectangular stress block under ultimate conditions, and the stability moment of resistance MR_s at the level of MB can be assumed to be provided by the axial load in the fin section acting at a lever arm about the centroid of the rectangular stress block as shown in Fig 31.



**(10) Allowable flexural compressive stresses, p_{ubc}
(taking into account slenderness (β) and material (γ_m))**

Before the stability moment of resistance MR_s can be compared with the assumed base moment (MB) of $\frac{W_k LH^2}{8}$, consideration must be given to the criteria affecting the allowable flexural compressive stresses, p_{ubc} , as this value dictates the stability moment of resistance. This is demonstrated in Fig 31, in which the mechanism producing the stability moment of resistance MR_s is shown.

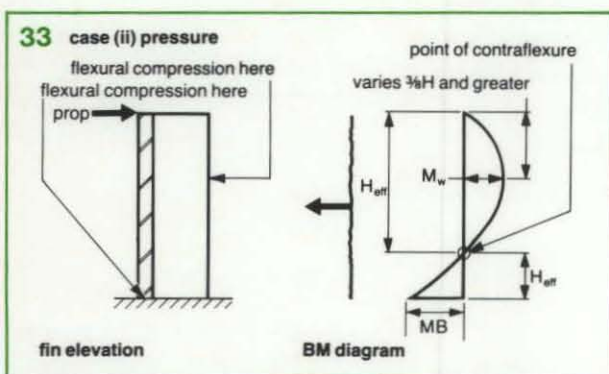
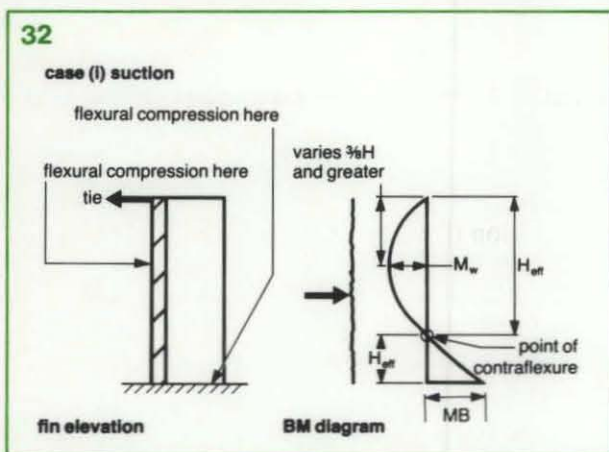
This flexural compressive stress can become significant and must be checked taking into account the tendency of the flange or fin to buckle at the point of application of the stress.

There is limited guidance given in BS 5628 on the effect of slenderness on the flexural compressive strength of masonry. This is because the flexural strength of masonry is assumed to be limited by the flexural tensile stresses – which is, perhaps, true of panel walls and the like, but not of the analysis of more complex geometric forms such as the fin wall.

The approach to the consideration of slenderness and flexural compressive stresses which follows is believed to provide a safe and practical design. It is expected that current research will allow more accurate analysis to be developed

Identification of problem:

Case (i) suction, showing zones of maximum values of flexural compression (Fig 32)

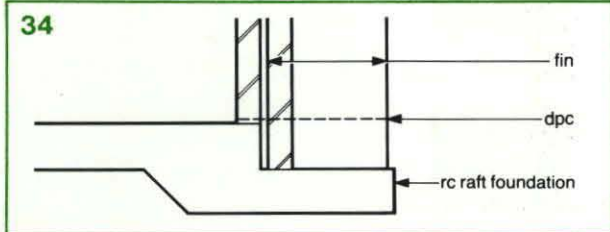


Case (ii) pressure, showing zones of maximum values of flexural compression (Fig 33)

Considering the wind suction loading case (i), flexural compression is applied to the flange of the T profile at the level of M_w . The buckling stability of the flange is provided by the projecting fin and, therefore, the effective length of the flange, for slenderness considerations, can be taken as twice the outstanding length of the flange from the face of the fin. Furthermore, if the flange is properly tied to the inner leaf of the cavity wall, the effective thickness of the flange, for slenderness considerations, can be taken as $\frac{2}{3}$ the sum of the thicknesses of the two leaves of the cavity wall.

Flexural compression is also applied to the end of the projecting fin at the level of MB. For this design example, the foundation is assumed to comprise a reinforced concrete raft slab as shown in Fig 34. The flexural compression applicable at this level is not influenced by slenderness considerations as the raft foundation can be assumed to provide full lateral stability.

Slenderness at this level would require careful consideration if the fin foundation was at a greater depth below ground level.



Considering the wind pressure loading case (ii), flexural compression is applied to the end of the projecting fin at the level of M_w .

The buckling stability of the fin cannot be considered to be fully provided by the flange of the Tee profile, as the flange is not of comparable lateral stiffness to the fin and would tend to rotate in attempting to prevent the fin buckling. Rather, it is considered that the slenderness of the fin should relate partly to its height and, as the full height of the fin would be over-cautious, it is proposed that the height between points of contraflexure would provide adequate safety. The effective thickness of the fin for slenderness considerations is taken as the actual thickness.

The design flexural compressive stress p_{ubc} can therefore be expressed as:

$$p_{ubc} = \frac{\beta f_k}{\gamma_m}$$

- where p_{ubc} = is the design flexural compressive stress
 β = capacity reduction factor derived from slenderness ratio
 f_k = characteristic compressive strength of masonry
 γ_m = partial safety factor for materials.

With the lateral restraint provided by the raft foundation at MB level, β can be taken as 1.0.

Therefore, $p_{ubc} = \frac{f_k}{\gamma_m}$ at MB level.

For this example,

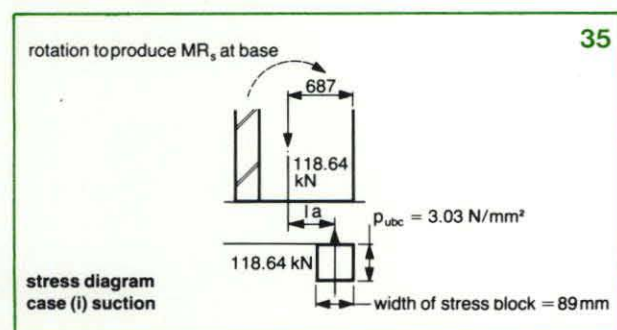
$f_k = 7.57 \text{ N/mm}^2$, based on 30 N/mm^2 bricks set in a designation (iii) mortar from BS 5628, Table 2a,
 $\gamma_m = 2.5$ as previously shown

Therefore:

$$p_{ubc} = \frac{7.57}{2.5} = 3.03 \text{ N/mm}^2$$

(11) Calculate MR_s and compare with MB

(a) Consider case (i) suction



From Fig 35, it is evident that the stability moment of resistance is provided by the flexural compressive stress at the end of the projecting fin, thus:

From Table 1, o.w. = $13.182 \text{ kN/m height}$

Therefore,

$$\begin{aligned} \text{design axial load in fin at MB} &= 0.9 \times 13.182 \times 10 \\ &= 118.64 \text{ kN} \end{aligned}$$

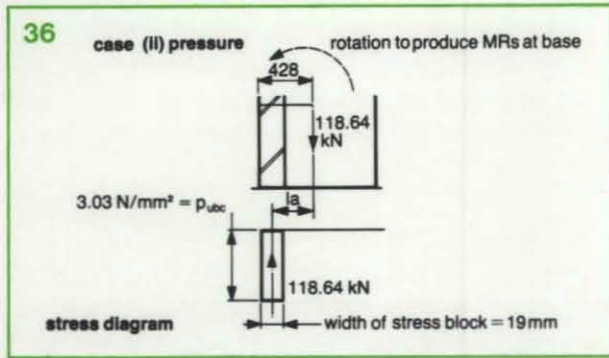
$$\begin{aligned} \text{Minimum width of stress block} &= \frac{\text{axial load on fin}}{\text{fin width} \times p_{ubc}} \\ &= \frac{118.64 \times 10^3}{440 \times 3.03} \\ &= 89 \text{ mm} \end{aligned}$$

$$\text{Lever arm} = 687 - \frac{89}{2} = 642.50 \text{ mm}$$

$$MR_s = 118.64 \times 0.6425 = 76.23 \text{ kNm}$$

As this is greater than MB = 37.05 kNm (see Fig 29) use MB in the design of the fin section.

(b) Consider case (ii) pressure



$$\begin{aligned} \text{Minimum width of stress block} &= \frac{\text{axial load on fin}}{\text{fin width} \times p_{abc}} \\ &= \frac{118.64 \times 10^3}{2084 \times 3.03} \\ &= 19 \text{ mm} \end{aligned}$$

$$\text{Lever arm} = 428 - \frac{19}{2} = 418.50 \text{ mm}$$

$$MR_s = 118.64 \times 0.4185 = 49.65 \text{ kNm}$$

The stability moment of resistance is shown to be less than:

$$MB = \frac{W_k LH^2}{8} = 54.15 \text{ kNm}$$

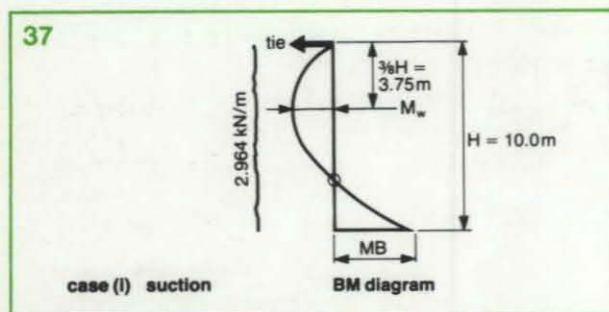
The base moment should therefore be limited to the value of stability moment, MR_s, 49.65 kNm and the bending moment diagram adjusted accordingly.

(12) Bending moment diagrams

Case (i) suction

$$M_w = \frac{9W_{k2}H^2}{128} = \frac{9 \times 2.964 \times 10^2}{128} = 20.84 \text{ kNm}$$

$$MB = \frac{W_{k2}H^2}{8} = \frac{2.964 \times 10^2}{8} = 37.05 \text{ kNm}$$



No adjustment is necessary to BM diagram as MR_s is greater than $\frac{W_{k2}H^2}{8}$ (ie MB) and therefore maximum M_w occurs at $\frac{3}{8}H$ from top of wall.

Case (ii) pressure

$$MR_s \text{ (calculated)} = 49.65 \text{ kNm}$$

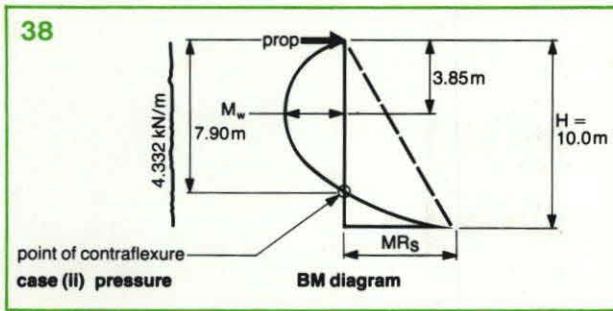
Find M_w from zero shear

$$\text{Prop} = \left(4.332 \times \frac{10}{2} \right) - \left(\frac{49.65}{10} \right) = 16.7 \text{ kN}$$

$$\text{Zero shear} = \frac{16.7}{4.332} = 3.85 \text{ m from top}$$

$$\begin{aligned} M_w &= \left(16.7 \times 3.85 \right) - \left(4.332 \times \frac{3.85^2}{2} \right) \\ &= 64.3 - 32.1 \\ &= 32.20 \text{ kNm} \end{aligned}$$

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Adjustment made to BM (bending moment) diagram to take account of MR_s being less than $\frac{W_{k1}H^2}{8}$ (ie MB) and therefore base moment limited to MR_s with M_w calculated by superimposing the free BM onto the stability moment produced by MR_s at base.

(13) Consider stresses at level of M_w

The stress considerations at the level of the maximum wall moment, assume triangular stress distribution, using elastic analysis, but relate to ultimate stress requirements at the extreme edges of the fin or wall face, depending on the wind direction considered. For compressive stress conditions, this gives a conservative solution.

Case (i) suction

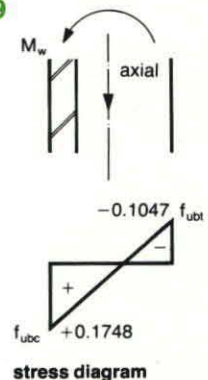
Properties of effective wall section from Table 1:

$$\begin{aligned} \text{o.w. of effective section} &= 13.182 \times 3.75 \\ &= 49.43 \text{ kN at level of } M_w \\ \text{Effective area} &= 0.6591 \text{ m}^2 \\ Z \text{ minimum} &= 0.12099 \text{ m}^3 \\ Z \text{ maximum} &= 0.19421 \text{ m}^3 \\ \text{Design axial load} &= \text{o.w. effective section} + \text{roof dead} - \text{roof uplift} \\ &= (\gamma_r \times 49.43) + \text{zero} \\ &= (0.9 \times 49.43) + \text{zero} \\ &= 44.49 \text{ kN} \end{aligned}$$

Flexural stresses at design load:

$$\begin{aligned} \text{Flexural compressive, } f_{ubc} &= + \frac{44.49 \times 10^3}{0.6591 \times 10^6} + \frac{20.84 \times 10^6}{0.19421 \times 10^9} \\ &= + 0.0675 + 0.1073 \\ &= + 0.1748 \text{ N/mm}^2 \\ \text{Flexural tensile, } f_{ubt} &= + \frac{44.49 \times 10^3}{0.6591 \times 10^6} - \frac{20.84 \times 10^6}{0.12099 \times 10^9} \\ &= + 0.0675 - 0.1722 \\ &= - 0.1047 \text{ N/mm}^2 \end{aligned}$$

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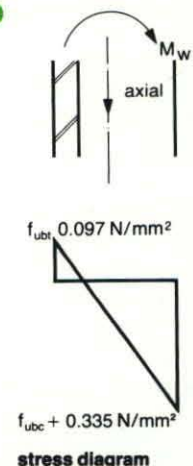
Case (ii) pressure

Properties of effective wall section from Table 1, as before, except that o.w. effective section at level of $M_w = 0.9 \times 13.182 \times 3.85 = 45.68 \text{ kN}$.

Flexural stresses at design load:

$$\begin{aligned} \text{Flexural compressive, } f_{ubc} &= + \frac{45.68 \times 10^3}{0.6591 \times 10^6} + \frac{32.2 \times 10^6}{0.12099 \times 10^9} \\ &= + 0.069 + 0.266 \\ &= + 0.335 \text{ N/mm}^2 \\ \text{Flexural tensile, } f_{ubt} &= + \frac{45.68 \times 10^3}{0.6591 \times 10^6} - \frac{32.2 \times 10^6}{0.19421 \times 10^9} \\ &= + 0.069 - 0.166 \\ &= - 0.097 \text{ N/mm}^2 \end{aligned}$$

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(14) Design flexural stress at M_w levels

(a) Design flexural tensile stress, p_{ubt}
(taking account of materials partial safety factor, γ_m)

$$p_{ubt} = \frac{f_{kx}}{\gamma_m} \text{ (from BS 5628, Clause 36.4.3)}$$

where $f_{kx} = 0.4 \text{ N/mm}^2$ for bricks with a water absorption of 7% to 12%
and $\gamma_m = 2.5$ as previously shown

$$p_{ubt} = \frac{0.4}{2.5} \\ = 0.16 \text{ N/mm}^2$$

By comparison with the f_{ubt} values calculated and shown in Figs 17 and 18, the wall is acceptable.

(b) Design flexural compressive stresses, p_{ubc}

$$p_{ubc} = \frac{\beta f_k}{\gamma_m}$$

Calculate respective β values for Case (i) and Case (ii) loadings at level of M_w .

Case (i) suction (flange in compression at M_w level)

$$\text{Slenderness ratio} = \frac{2 \times \text{flange outstanding length}}{\text{effective thickness}} = \frac{2 \times \left(\frac{2084 - 440}{2} \right)}{\frac{2}{3} (102.5 + 102.5)} \\ = 12$$

The stressed areas can be considered as axially loaded,

therefore $e_x = 0$

therefore for SR = 12 and $e_x = 0$, from BS 5628, Table 7, $\beta = 0.93$

$$p_{ubc} = \frac{0.93 \times 7.57}{2.5} \\ = + 2.82 \text{ N/mm}^2$$

Case (ii) pressure (end of fin in compression at M_w)

$$\text{Slenderness ratio} = \frac{\text{effective height between points of contraflexure}}{\text{actual thickness}} \\ = \frac{7900}{440} \text{ (see Fig 38)} \\ = 18$$

Therefore, $\beta = 0.77$ from BS 5628, Table 7

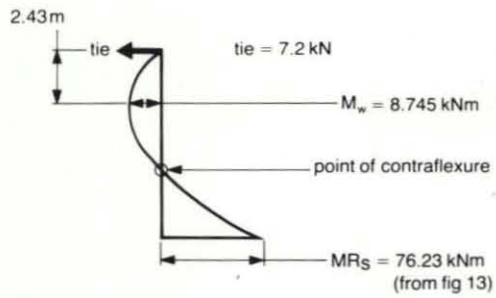
$$p_{ubc} = \frac{0.77 \times 7.57}{2.5} \\ = + 2.33 \text{ N/mm}^2$$

By comparison with the f_{ubc} values calculated and shown in Figs 39 and 40, the wall is acceptable.

(15) Consider fins with deflected roof prop

It is evident that the deflection of the roof wind girder induces additional rotation at the level of MB.

In this design example, the MR_s limited the moment at the base under wind pressure loading, and the additional rotation will not alter the design bending moment diagram shown in Fig 38. The base moment for wind suction loading, when the roof support does not deflect, is $\frac{W_{k2} H^2}{8}$ (Fig 37). But, as the deflecting roof support induces further rotation at base level, the section cracks and takes full advantage of the stability moment of resistance MR_s . The revised design bending moment diagram for this condition, when compared with Fig 37, is shown in Fig 41. The reduced wall moment value is obviously acceptable, whilst the increase in the moment at base level is also shown (Fig 35) to be acceptable. However, this should be fully checked if slenderness reductions are applicable at this level.

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SUGGESTED DESIGN PROCEDURE

After some experience, a competent designer will be able to shorten the design process considerably.

A suggested design procedure is as follows:

- (1) Calculate wind loadings.
- (2) Calculate dead and imposed loadings.
- (3) Assess critical loading conditions.
- (4) Select trial section.
- (5) Calculate stability moments MR_s at base.
- (6) Calculate position of maximum wall moments.
- (7) Calculate magnitude of maximum wall moment M_w .
- (8) Check compressive stress at base level.
- (9) Check loadings and stresses at levels of M_w .
- (10) Select brick and mortar strength required.

- (1) **Brick diaphragm walls in tall single-storey buildings.** Curtin & Shaw. Brick Development Association.
- (2) **BS 5628: Part 1: 1978, The structural use of masonry.** British Standards Institution.
- (3) **CP 111: 1970, Structural recommendations for loadbearing walls.** British Standards Institution.
- (4) **CP 121: Part 1: 1973: Walling, Brick and block masonry.** British Standards Institution.
- (5) **Energy conservation. Thermal insulation of brick buildings complying with Part FF of the Building Regulations.** Brick Development Association.
- (6) **Thermally insulated brickwork.** BDA Design Note No 2. Brick Development Association.
- (7) **BS 743: 1970, Materials for damp proof courses. Metric units.** British Standards Institution.
- (8) **BS 1243: 1978, Specification for metal ties for cavity wall construction.** British Standards Institution.
- (9) **CP 3: Chapter V: Part 2: 1972, Wind loads.** British Standards Institution.

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