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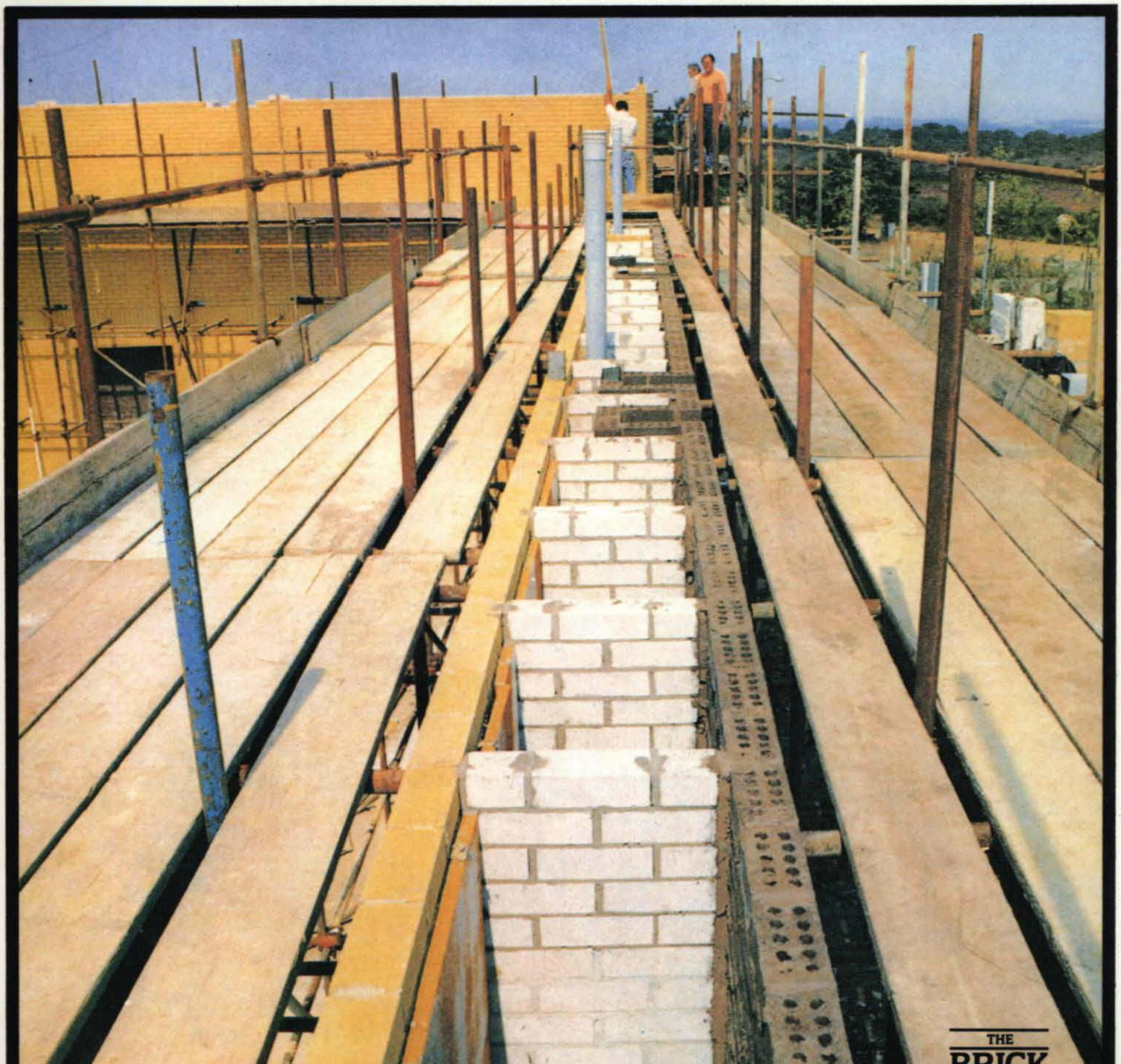
DESIGN OF BRICK DIAPHRAGM WALLS



THE BRICK DEVELOPMENT ASSOCIATION

Edited by J Morton BSc PhD CEng MICE M/Chem MCIM

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Design of brick diaphragm walls

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The Brick Development Association

CONTENTS

INTRODUCTION AND GENERAL ARRANGEMENT

1

INTRODUCTION

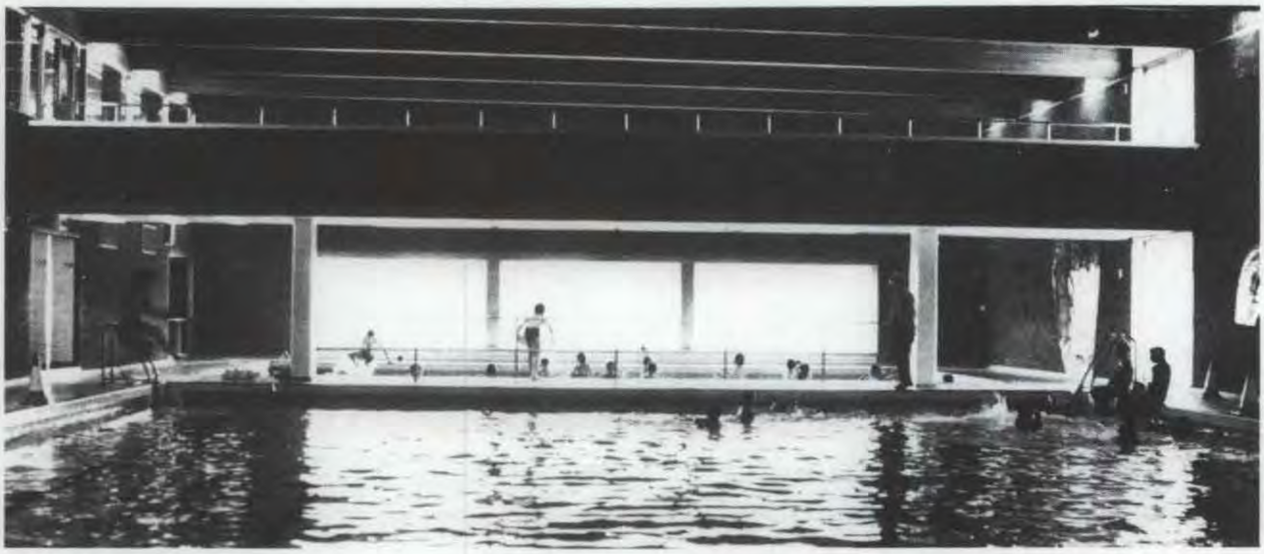
Tall, single-storey structures enclosing large open areas account for a large number of the buildings constructed in this country and abroad. They include sports and assembly halls, warehouses, theatres, gymnasiums, garages, churches, squash courts, workshops, supermarkets, stadiums and industrial units.

Traditionally, the vast majority of these structures have their roofs supported on steel columns. The columns are then enveloped by a cladding material, which often requires secondary supporting steelwork and, on occasions, the cladding is backed up by an insulation barrier which, in turn, is protected on the inner face by a hard lining. The steel columns invariably require some degree of fire protection and their protection from corrosion is related to the life expectancy and degree of exposure.

The resulting 'wall' thus requires between four and six different materials and several sub-contractors, suppliers and trades. Apart from the frame itself, the cladding and lining require periodic maintenance and lack the durability and aesthetic qualities of brickwork. Vandal-resistance is a further bonus of brickwork's durability and robustness.

Brick diaphragm walls form the structure, cladding and lining in one material, using only one trade carried out by the main contractor and can be insulated to any required level. The authors' experience has shown that brick diaphragm walls are well suited to the building types listed earlier, and have proved to be more

Below *An early example of diaphragm wall construction. Gymnasium, Wellfield School, Leyland. Walls are only 350 mm thick. Architects: Fairbrother, Hall & Hedges. Structural engineers: W. G. Curtin & Partners.*



economical, speedier and simpler to construct, and more durable than the traditional steel frame and sheeted cladding.

Brickwork, like any other structural material, requires an understanding of its properties in order to use it economically. Whilst possessing extremely high resistance to compressive stresses, brickwork has relatively low resistance to tensile stresses and, therefore, it becomes important when resisting bending stresses to (a) use a high $\frac{Z}{A}$ ratio, and (b) to take advantage of the gravitational forces involved. Both these requirements involve a similar geometric distribution of materials: that is, to provide the material at its largest practical lever arm position. It is also necessary to provide adequate resistance to shear forces and buckling of the compression zone.

From the practical point of view, the geometrical arrangement of the wall should also conform to standard brick dimensions. By using a minimum thickness of brick skin and applying the above principles, diaphragm wall construction was evolved and developed.

GENERAL ARRANGEMENT AND DETAILS

A diaphragm wall comprises two parallel leaves of brickwork joined by perpendicular brick cross-ribs (or diaphragms), bonded at regular intervals to form 'box' or I sections, see figure 1.

The two parallel leaves of the wall act as flanges in resisting bending stresses, and are stiffened by the cross-ribs which act as webs to resist the shear forces. To keep costs and space to a minimum, the width between flanges is designed to suit the individual requirements of each project.

Above *Diaphragm wall construction has been developed to meet the needs of much wider and taller structures than the Leyland gymnasium. Detail of swimming pool, Oval Sports Centre, Bebington. Architects: Cheshire County Architects Department. Structural engineers: W. G. Curtin & Partners.*

Diaphragm wall construction becomes more and more economical as the height of the wall increases. However, recently, narrow diaphragm walls with a half-brick wide cavity have proved to be economical alternatives, both in construction time and financial terms, to the more traditional steel portal frame structures for buildings with wall heights of about 4.5 m. On the other hand, diaphragm walls have little advantage where normal cavity brickwork can meet all the structural requirements. To date, buildings with wall heights of up to 10 m have been designed by the authors, and there is no reason to suppose that this is anywhere near the structural and economic limit.

Roof and capping beams

In order to obtain the greatest economy in the total cost of the structure, the roof of diaphragm wall structures should be used, when possible, as a horizontal plate member to prop and tie the tops of the walls and transfer the resulting horizontal reactions to the transverse walls which act as shear walls.

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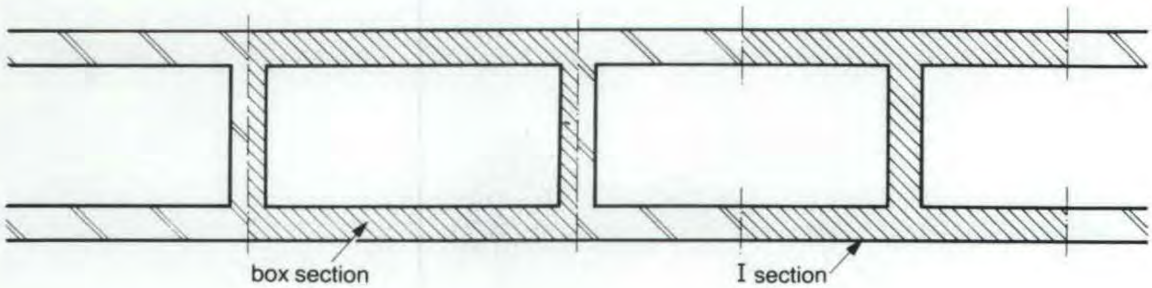
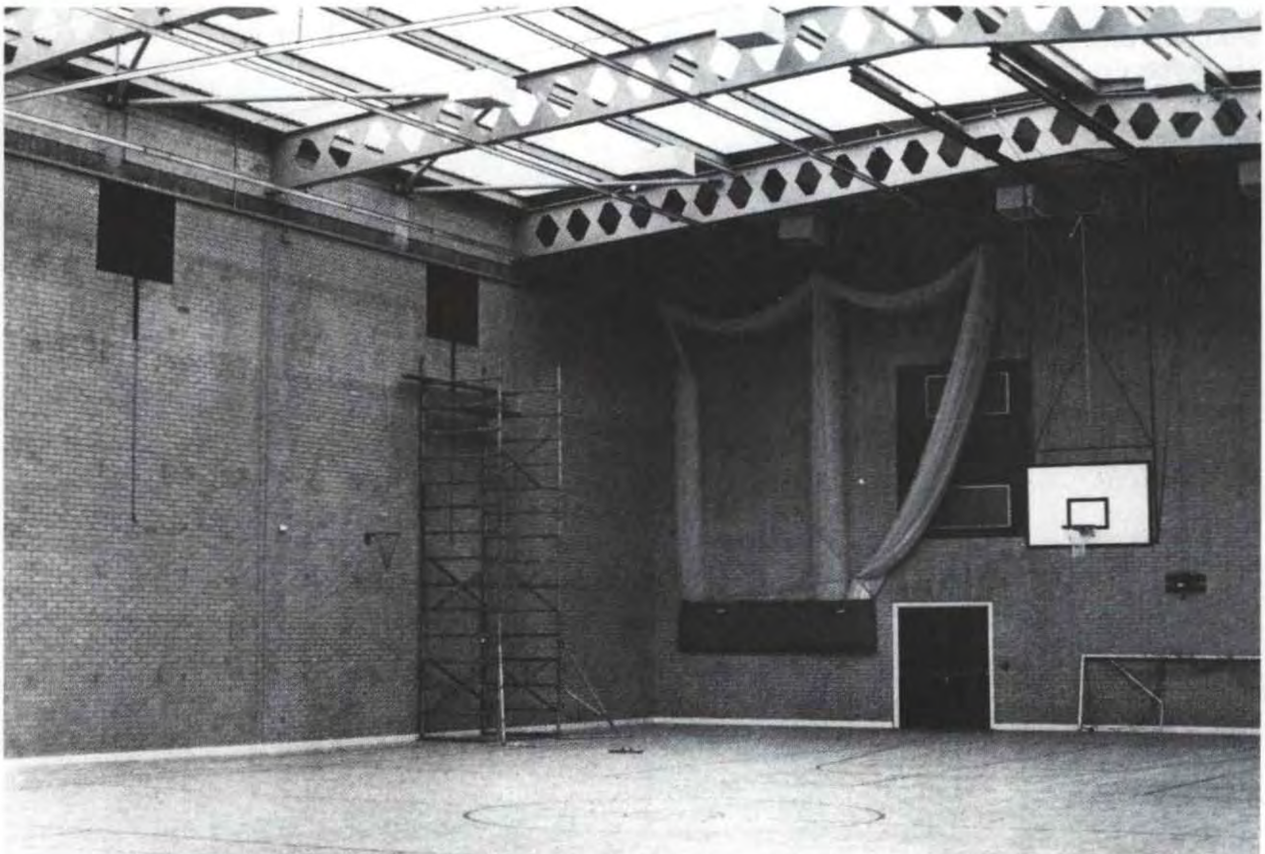
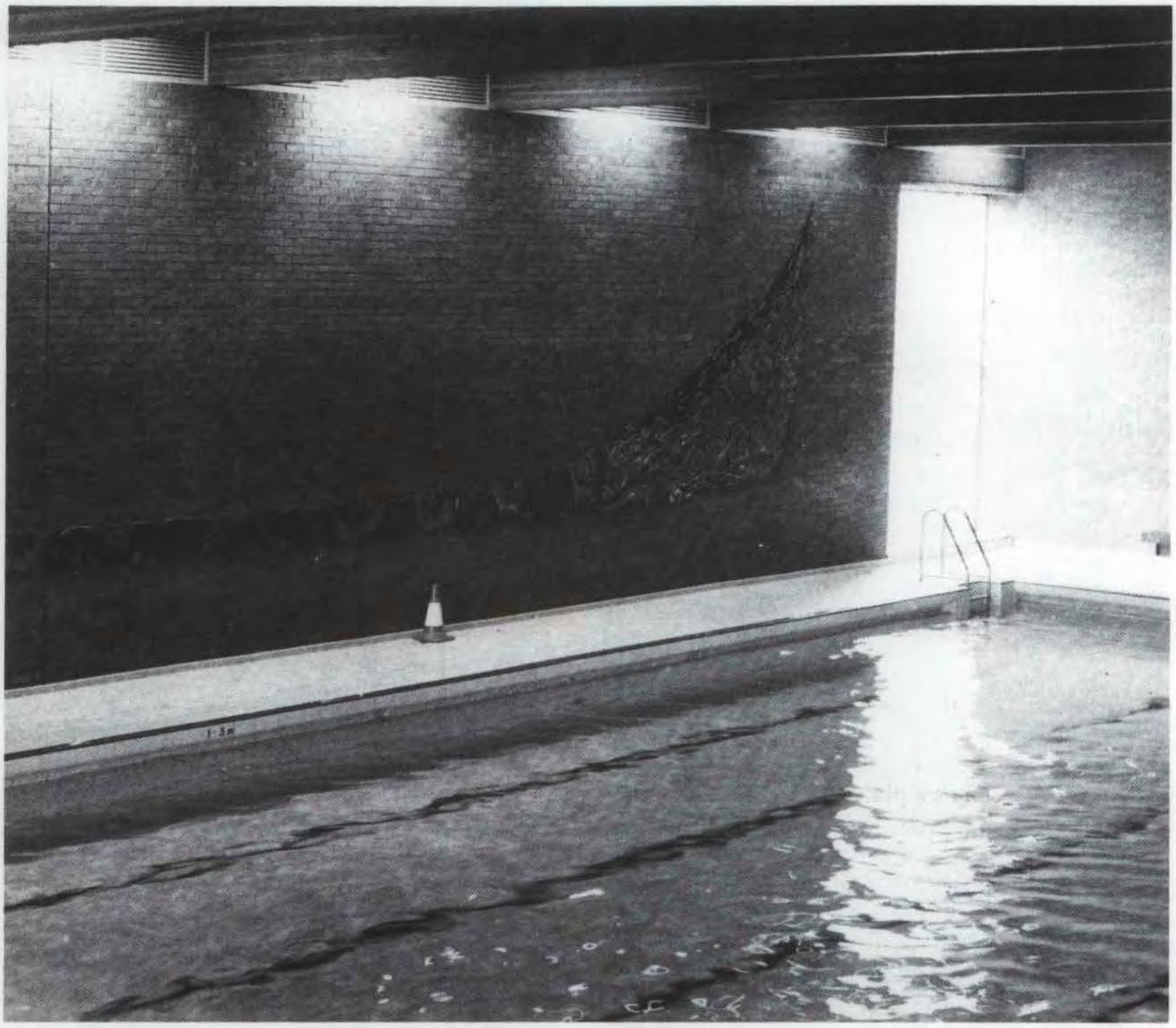


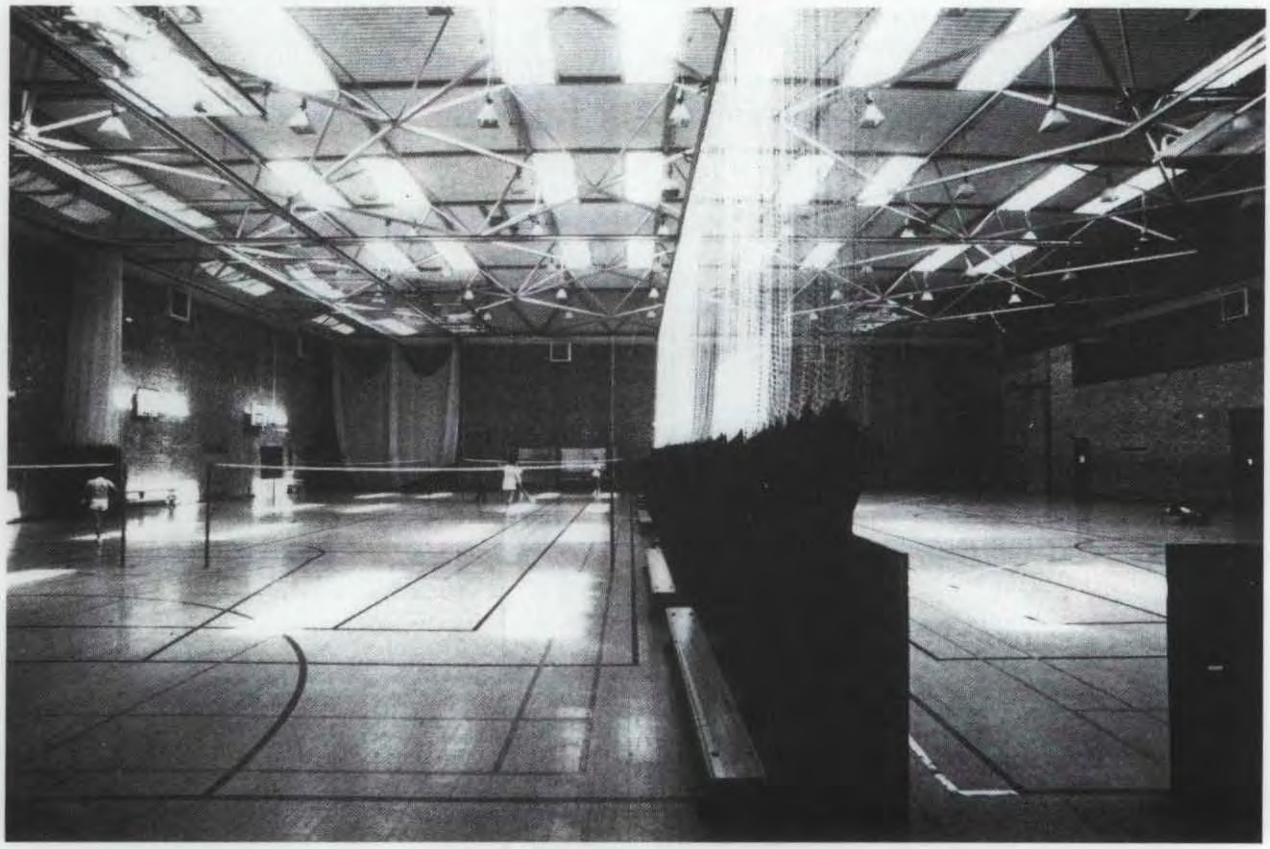
Below *Gymnasium, Leyland. Roof construction is steel universal beam sections carried on padstones at 3.6 m centres, with timber 'A' frames spanning between the steelwork.*

Right *Swimming pool, Turton School, Bolton. Roof structure is pre-cast concrete beams at 6 m centres supporting a domed roof light.*

Below *Sports hall, Tomlinson School, Kearsley. Roof construction is castellated steel beams at 6 m centres, with pressed steel purlins supporting double skin PVC sheeting. CHS section wind girders.*







Top *Oval Sports Centre, Bebington. Roof is laminated timber beams at 3.6 m centres with solid timber decking.*

Above *Sports hall, Sutton High School, St Helens. Roof construction consists of a space deck of lightweight tubular steel sections with metal decking and rooflights over.*

A capping beam can be used on top of the diaphragm wall to transfer these forces into the roof deck and to overcome uplift forces from wind suction acting on lightweight decking. If necessary, the beam can also be used as the boom member of the roof plate. The roof deck can be of a variety of materials and supported in many ways. Depending on the spans involved, the most economical roof beams may be universal beams, castellated beams or lattice girders, which can be spaced at centres to suit the most economical arrangement, taking into account the selected decking material. Solid whitewood decking on glulam beams has also been used as a horizontal plate propping the head of the wall. Whilst achieving an improved aesthetic internal finish and freedom from corrosion in swimming pools and similar buildings, this solution is considerably more expensive than the steel alternatives. On long spans, a space deck can prove to be more economical in providing the necessary decking support.

A space deck can also act as a suitable plate to transfer the propping loads to the transverse walls. Often, the decking material, if suitably fixed, can be used as a plate in conjunction with the main roof beams. But where this is not the case, a horizontal girder can be incorporated using the concrete capping beams as boom members.

The capping beam at roof level can be constructed by using either in-situ concrete (on a bridge shutter of asbestos or similar material) or by precasting the beam in bay lengths and using a suitable connection to transfer the forces at the joints. The capping beam is used as the seating and fixing for the roof structure, as shown in figure 2. Probably the more successful method of constructing a capping beam is that of precasting, since this overcomes the problems of keeping the facing bricks clean, and the expense of the permanent shutter which may be necessary for the in-situ solution. For in-situ beams, the shuttering can be retrieved by leaving one of the wall leaves down approximately four courses and building up later.

Foundations
At foundation level, the pressures are so low with this form of construction that the use of a nominal strip footing is usually adequate, but this must, of course, be determined from consideration of the ground conditions (figure 3). The designer should remember to include in the foundation design the effect of the applied moment at the base of the diaphragm wall.

Above *Reinforcement for an in situ capping beam. Sports hall, Ormskirk.*

Openings in walls
Large door and window openings can create high local loading conditions from the horizontal wind loading and increased axial loads at beam bearings. The openings can be dealt with by providing a beam or lintel to carry the vertical load, and by using extra ribs or thicker ribs on each side of the openings (figure 4). Vertical dpcs should be provided at external openings.

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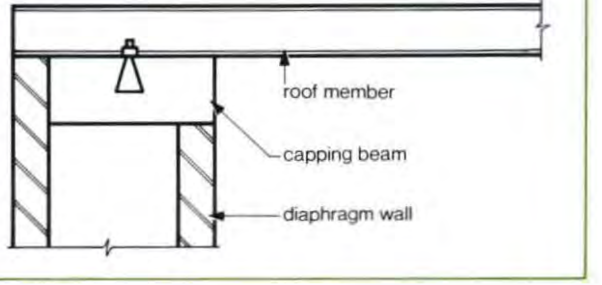

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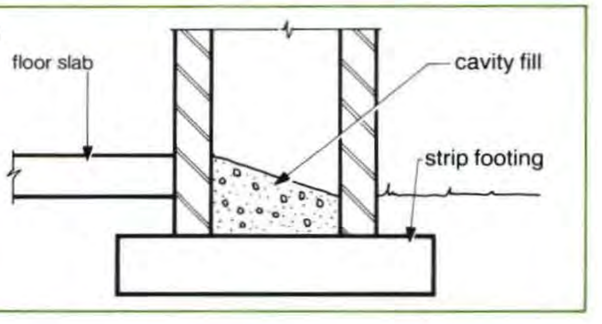

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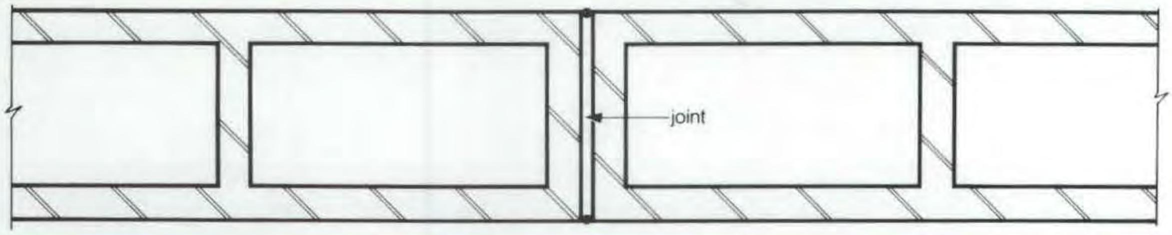

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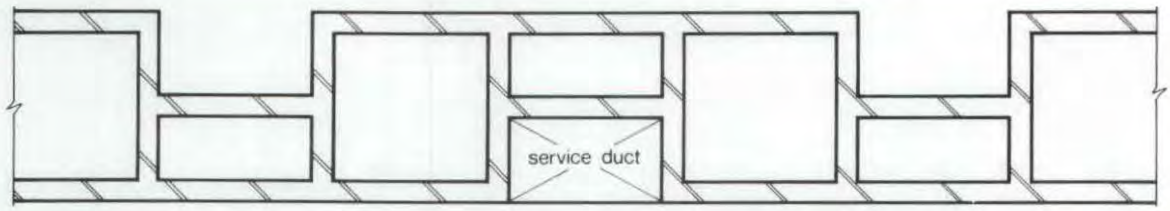

Left *Extract fans accommodated in the voids. Swimming pool, Turton School.*

Above *Diaphragm wall with board insulation (insulation shown is less than would normally be required).*

Joints

Movement joints are required at the appropriate centres, in accordance with the normal recommendations for brickwork given in other BDA technical publications and in CP 121. Joints can easily be accommodated by providing double ribs, one at each side of the joint (figure 5).

Services

Because of the large voids within diaphragm walls, it is possible to accommodate certain services within them. Checks must be carried out to ensure that the location and size of the access holes do not cause local overstressing of the brickwork (bearing in mind that measures can be adopted to cater for such a condition). In addition, access to services and the possibility of corrosion must be given full consideration if maintenance costs are to be minimised. Service ducts can be incorporated in the wall (figure 6). Such ducts should, of course, be ventilated when housing gas pipes (for further information on ventilating diaphragm wall voids see 'Rain resistance' and 'Damp proof courses').

Sound insulation

Brick diaphragm walls will not often be used in positions where there is a statutory requirement for sound insulation. But, there may well be situations where it is desirable to use the walls as a sound barrier against external noise. Although no tests have been carried out, on the basis of the mass law, the diaphragm wall will have at least as good a performance as a traditional cavity wall of the same materials. However, it is believed that, in practice, the sound insulation of the diaphragm wall would be better, due to the wider cavity.

Thermal insulation

It has been generally accepted that the U-value of diaphragm walls may be assumed to be 10% higher than for conventional cavity walls built of the same materials. This loss of performance is due to air circulation in the large voids, and the direct brickwork connection between inner and outer leaves. However, also owing to the large voids, it is very simple to improve the thermal insulation value of a diaphragm wall. The Building Research Establishment has carried out extensive theoretical analyses of heat flow through diaphragm walls*, and has confirmed the view

* *The thermal insulation of diaphragm walls – an investigation of U-values and surface temperatures using a 2-dimensional heat flow model. T. I. Ward, Building Research Establishment. The International Journal of Masonry Construction, Vol 1, No 3, 1981.*

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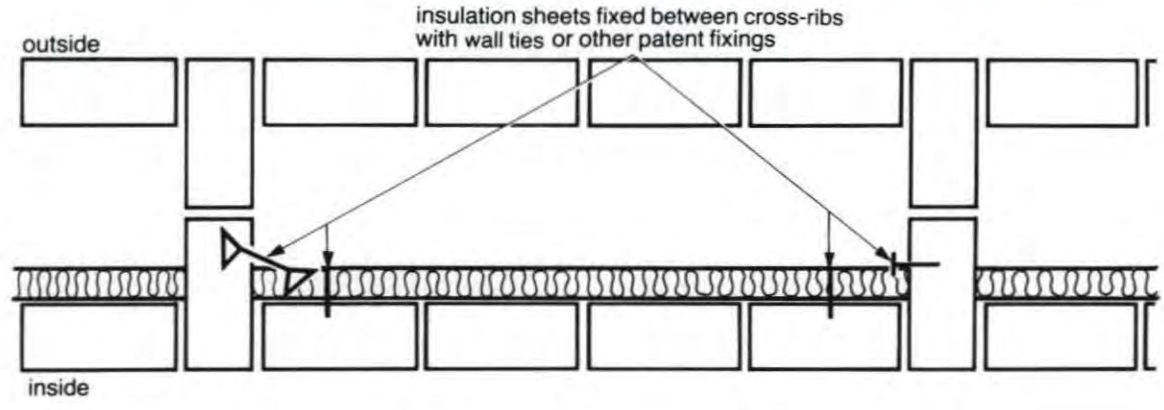

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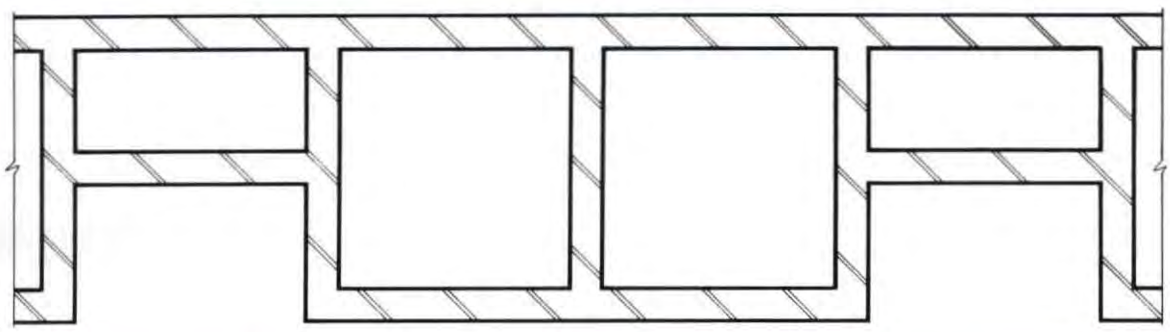


that the requirements of Part FF of the Building Regulations can be met by fixing insulating batts of 75 mm or 100 mm thickness to the inner leaf of the diaphragm walls (figure7).The actual thickness would depend on the insulating material and the particular bricks used. Further improvement of the U-value can be achieved by completely filling the void with insulant.

The Building Research Establishment research also confirmed that the dew point will always occur in the cross-rib or void, and that condensation or pattern staining on the inside surface adjacent to the cross-ribs or diaphragms is very unlikely to occur. This view concurs with observations of the actual performance of various buildings over a 12-year period.

Rain resistance

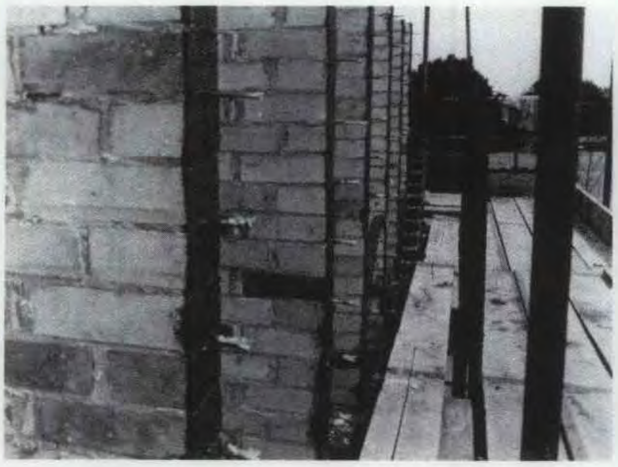
Some designers have expressed concern that driving rain might traverse the cross-ribs (or diaphragms) and result in damp penetration on the inner face of the walls. Examination of existing buildings, many of which are situated in exposed locations in the north west of England, has shown no sign of rain penetration. However, a series of tests, using Fletton bricks, has been carried out on full scale diaphragm walls (two bricks total width) in accordance with BS 4315: Part 2:1970. One test, which may be considered to have given conditions similar to 63 mm of rain falling in a twenty-four hour period with a constant 27 m/s wind, showed no sign of water penetration until the fifth day when a damp patch was recorded on a header. These rain and wind conditions, of course, are far in excess of real conditions and, therefore, the tests support the experience that rain will not penetrate diaphragm walls of this thickness. It is considered that, under normal conditions, the dampness in the cross-ribs should dry out before reaching the inner leaf, and to this end, benefits can be derived from ventilating the voids.

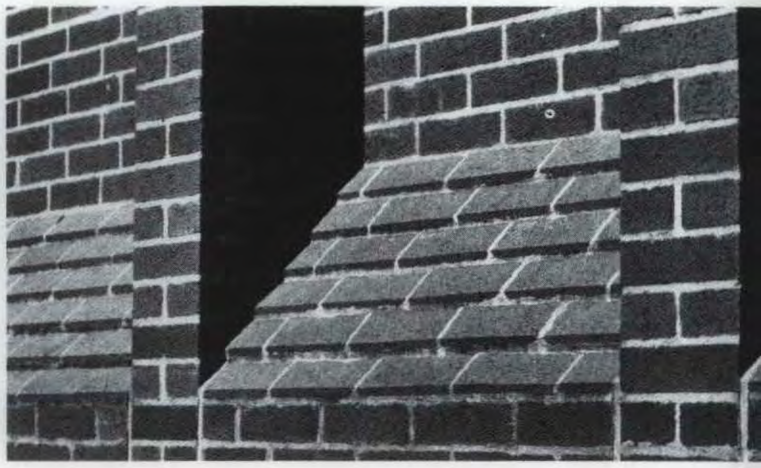
More recently, as has been said, diaphragm walls with a half-brick wide cavity have proved extremely economical for industrial units with 4.5 m high walls. The authors consider that, in these situations, damp penetration might become a possibility and recommend the use of tied cross-ribs using stainless steel shear ties and a vertical DPC (possibly brush applied).

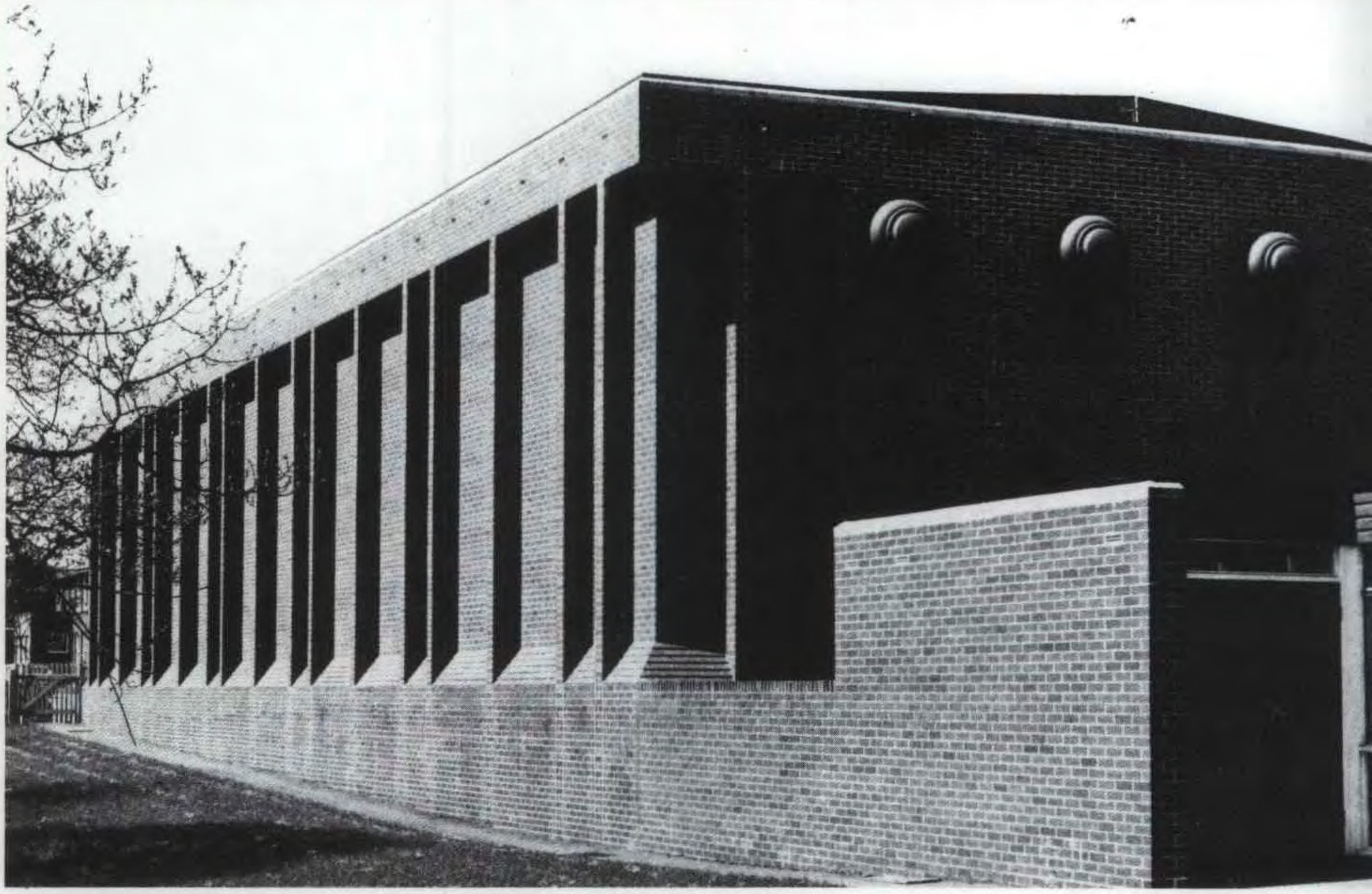
ARCHITECTURAL DESIGN

The junction between the wall and roof of the building can be treated in many different ways and some examples are illustrated. It is not essential for the diaphragm wall to be designed with flat faces on each side and, particularly on tall buildings, a fluted arrangement can be neatly incorporated in the structure (figure 8) thus creating more interesting elevations.

Depending on the cost implications and appearance requirements, it is possible to use either bonded joints between the cross-ribs and leaves, or butt joints with designed shear ties. Structurally, it is preferable to use bonded joints



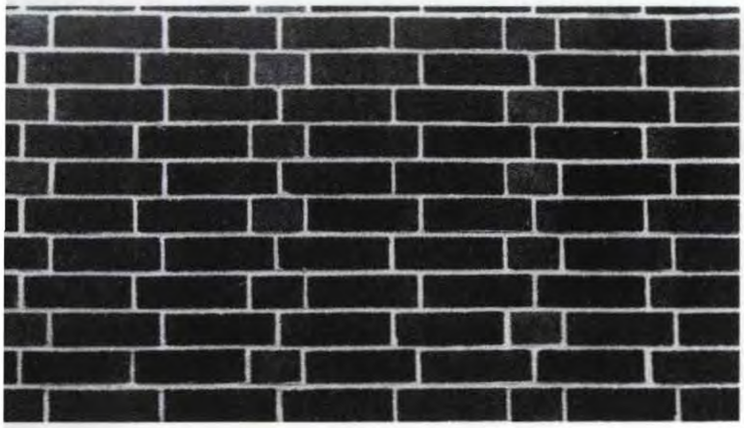




Top left *Shear ties and a brush applied vertical dpc. Corfe Hills Upper School.*

Top right and middle *Swimming pool, Turton School, Bolton. Cross-ribs expressed externally. Architects: C. B. Pearson, Son & Partners. Structural engineers: W. G. Curtin & Partners.*

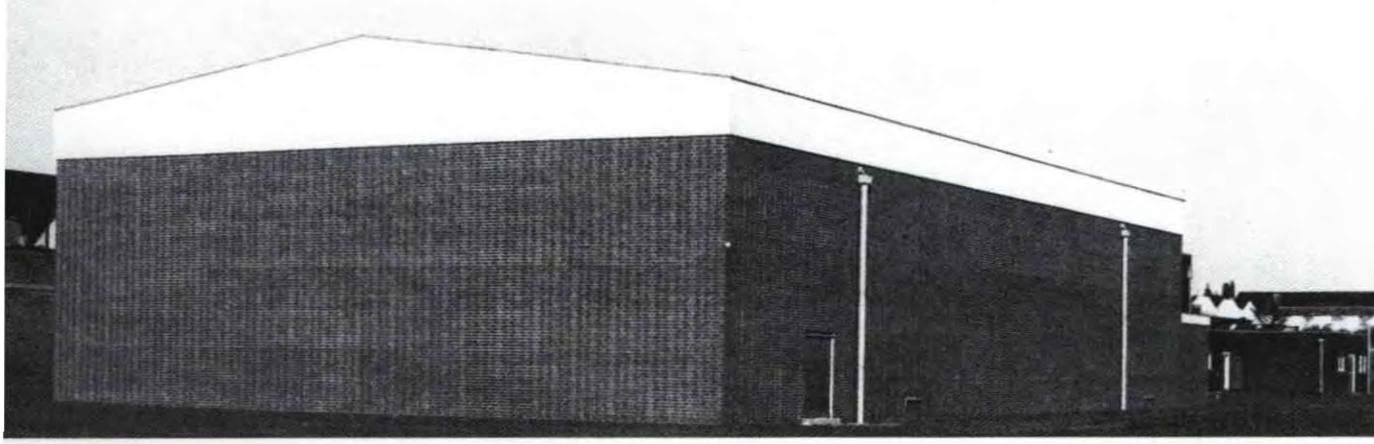
Above and right *Sports hall, Tomlinson School, Kearsley. Castellated effect above capping beam. Architects: Hall & Wilson. Structural engineers: W. G. Curtin & Partners.*

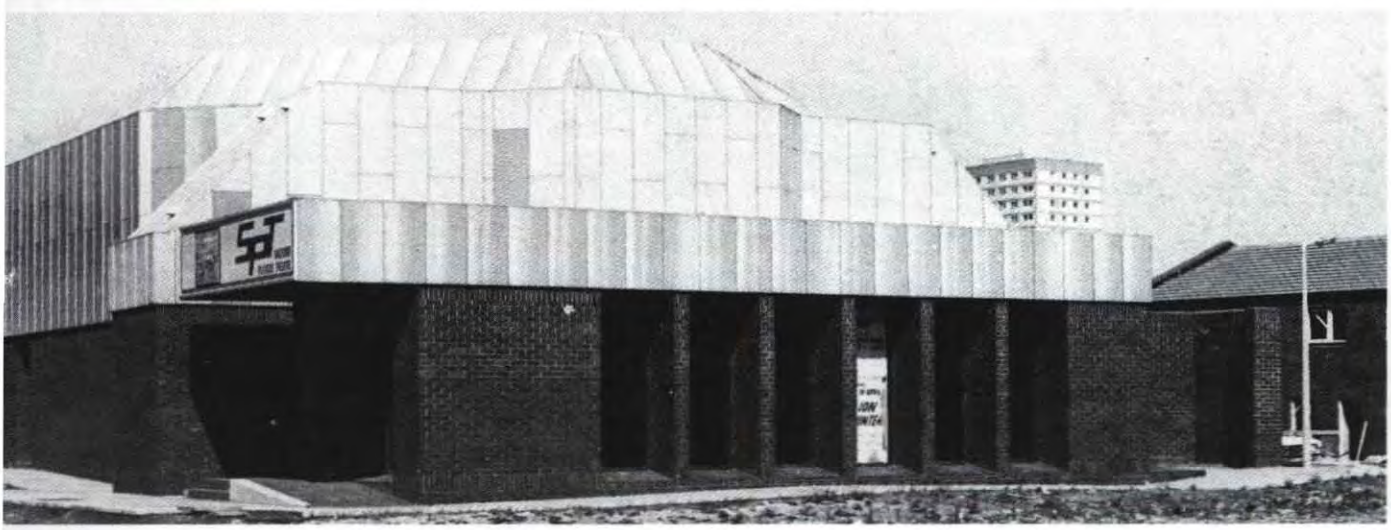


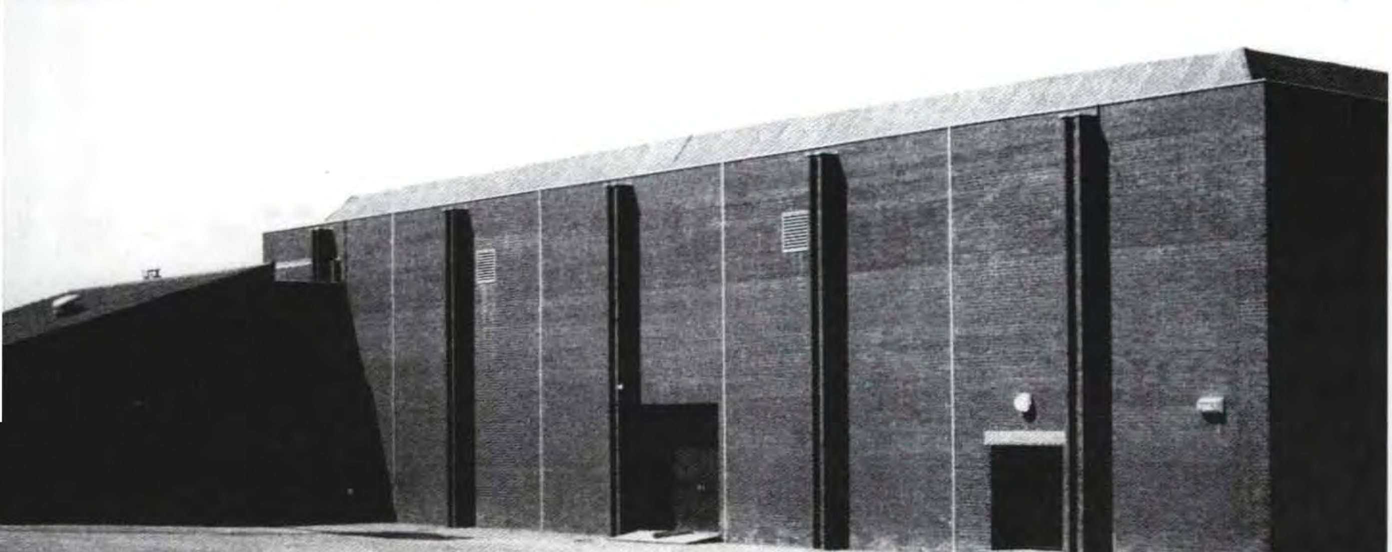
Left and below *Sports hall, St Gabriel's School, Bury. False headers between cross-ribs intensify the bond pattern. Architects: Richard Byrom, Hill & Partners. Structural engineers: W. G. Curtin & Partners.*

Middle *Salford Players Theatre. Architects: Wilson & Womersley. Structural engineers: W. G. Curtin & Partners.*

Bottom *Sports hall, Sutton High School, St Helens, is 10 m high, and 37 m square on plan. Rainwater pipes could have been accommodated within the voids of the diaphragm walls. Designers preferred to locate them externally, enclosed in brickwork, and to use the resulting projections as an architectural feature to break down the visual mass of the walls. Architects: W. & J. B. Ellis. Structural engineers: W. G. Curtin & Partners.*







which lead to headers being visible on the faces of the brickwork. The headers can be used as a feature and can incorporate a different coloured brick to that of the stretchers. In some cases, the labour costs for providing bonded brickwork have been more than those for unbonded joints containing shear ties. However, the economics of this appear to vary from job to job. If unbonded cross-ribs are used, the shear ties must be designed to take the vertical shear forces and must be sufficiently durable to resist corrosion.

A variety of facing bricks can be used to create patterns in diaphragm wall construction, and the bricks used in the ribs need not necessarily be the same as in the flanges. However, if different types of bricks are combined in the same construction, consideration must be given to their compatability with regard to thermal and moisture movements. High strength is seldom a significant criterion for bricks in diaphragm walls.

CONSTRUCTION

Cavity cleaning

The problem of mortar droppings etc, within the voids of the walls becomes less as the width of the void increases and, in most cases, elaborate methods of cleaning out the voids are not necessary provided that normal care is taken during construction.

Temporary propping

Like most other walls, the diaphragm wall is in a critical state during erection, prior to the roof being constructed and fixed. During this period, the contractor must take the normal temporary precautions such as propping the walls with the bricklayers' scaffolding or other means, to ensure that they remain stable. Greater accuracy and better workmanship is achieved with the use of scaffolding on both sides of diaphragm walls, rather than working overhand from one side. The double scaffold system, which is generally recommended by the authors, can be adapted and used to provide adequate temporary propping to the walls.

Damp proof courses and membranes

Horizontal damp proof courses should be selected to give the necessary shear resistance to prevent sliding, and should not squeeze out under the vertical load. Vertical damp proof membranes between leaf and cross-ribs are not normally required in diaphragm walls of 2 bricks width.

Where such vertical damp proof membranes are considered advisable (for example, in narrower diaphragm walls, and/or particularly exposed locations) it is essential that they should not prevent the tying of the cross-ribs to the leaf. Brush-applied types of damp-proofing membranes in conjunction with non-ferrous shear ties have

Above and right *St Martin de Porres Church, Luton. Conflicting geometry of the walls and the roof created a structural problem. Making the walls and the roof structurally independent of each other, left the 7.7 m high walls surrounding the sanctuary standing as unpropped cantilevers and vulnerable to heavy wind loads. Diaphragm wall construction overcame this difficulty. Internally, full height windows set in the deep reveals of the diaphragm walls provide subdued yet dramatic natural lighting. Architects: Ellis Williams Partnership. Structural engineers: W. G. Curtin & Partners.*

Below right *Detail of workshop and stores building, Howley Park Brickworks, Dewsbury. Panels of conventional brickwork introduced at external corners provide visual interest and depth of modelling. Architect: S. E. Bell, DipArch RIBA (Head of Technical Services, George Armitage & Sons Ltd). Structural engineers: W. G. Curtin & Partners.*

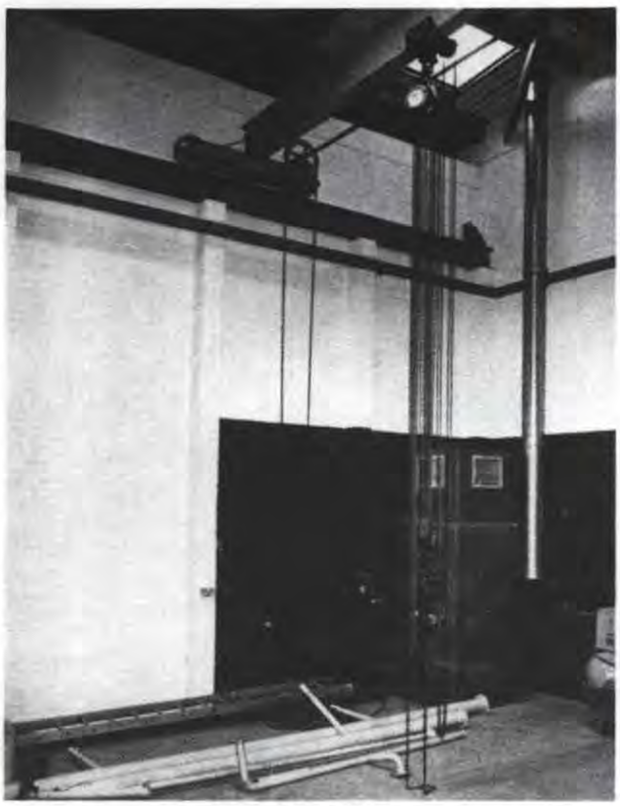
proved to be a successful combination in practice. The quality of the shear ties should logically reflect the life-expectancy of the structure. Fully filled smooth mortar joints are essential to the efficiency of brush-applied dpms, and the quality of the workmanship to achieve this in such areas is of paramount importance.

When bonded cross-ribs are used, the authors consider that the capacity of the brickwork to absorb moisture may be a significant factor in its resistance to rain penetration, with a greater capacity to absorb moisture leading to improved rain resistance qualities of the total wall. Benefit can be derived from introducing ventilation by means of air bricks at the top and bottom of the wall with, perhaps, perforations introduced into the cross-ribs to avoid the need of venting every cell. Account must be taken of such perforations in the design of the shear resistance of the ribs.

STRUCTURAL DESIGN

The main calculation involved in the design of diaphragm walls is for the critical condition of combined dead and wind loading. This takes into account the maximum uplift and maximum flexural wind stresses. The compressive stresses involved when the combined dead, superimposed and wind loading is applied are, in general, so low that the selection of a suitable brick and mortar is based mainly on flexural resistance and the minimum requirements for durability and absorption.

Calculations are carried out on a trial and error basis, by adopting a trial section and then checking the stress conditions. For a more detailed discussion and worked examples see page 17 onwards.



Above *Detail of workshop interior, Howley Park Brickworks. Crane gantry rails are supported on 215 mm square piers bonded into the diaphragm walls at alternate rib positions.*

EXPERIENCE AND PERFORMANCE OF DIAPHRAGM WALLS

Since the publication in 1977 of the first edition of this guide, many diaphragm walls have been constructed throughout the country, and the rapid expansion of interest which they have generated has shown the technique to be both technically acceptable and functionally commendable.

The buildings already constructed have survived:
(a) the highest wind gust speeds recorded in the UK;

(b) the hottest summer on record with air temperatures of over 30°C and external wall temperatures of 45°C;

(c) the wettest autumn on record in the UK;

(d) one of the most severe winters this century.

These buildings have performed successfully and no problems have developed as a result of the construction method.

ECONOMICS

The most cogent reason for adopting any particular building method is that it is economical. This has always been the prime factor for adopting the diaphragm wall method of construction in the buildings where it has or is being used. There is considerable evidence that, for the more sophisticated type of structure such as sports halls, theatres, swimming pools, etc, the diaphragm wall structure is the most economical.

More recent investigations indicate that, for basic industrial structures, this type of building can also provide the most cost-effective solution. Experience has shown that diaphragm wall buildings can show a considerable saving of time both in the pre-contract period and particularly during construction itself. This is because the design, estimating and tendering procedures are simplified and, during construction of the walls, elimination of dependence on a variety of materials, sub-contractors and separate site operations, enables the general contractor to work to a tighter schedule under his own close control.

OTHER APPLICATIONS

Although the diaphragm wall was developed for tall single-storey, wide-span, structures, it has applications in other fields, particularly where lateral loading is more significant than vertical loading. For example, a diaphragm has been used as a mass-retaining wall on a site which had a large amount of demolition rubble. The rubble was used to fill the cavity, and a cheap and strong mass-retaining wall was achieved. The wall was constructed in 1970 as part of a landscape development (figures 9 and 10).

Diaphragm walls could also be used as sound reflectors on motorways in urban areas. At the present moment, some reflectors are going up in steel, pre-cast concrete and timber, and it is thought that brick diaphragm walls would be cheaper, certainly more durable, and have greater aesthetic appeal. They may also be used for fire barriers in industrial buildings, and for farm silos for the storage of grain, potatoes, etc.

Diaphragm walls were initially used almost exclusively for sports halls in schools and leisure centres where their popularity has inspired architects, engineers, developers and contractors to apply the technique to numerous other types of buildings. Amongst these are now included factories, warehouses (with and without overhead travelling cranes), garages, churches, sports halls, theatres, assembly halls, squash courts, retaining walls.

Diaphragm walls, designed to act compositely with the foundations, and post-tensioned to minimise in-plane tensile stresses, were used on a project which was subject to massive ground

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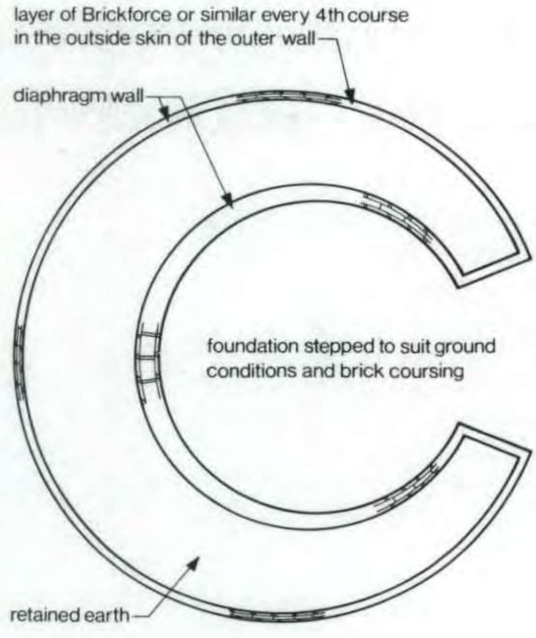

plan of site

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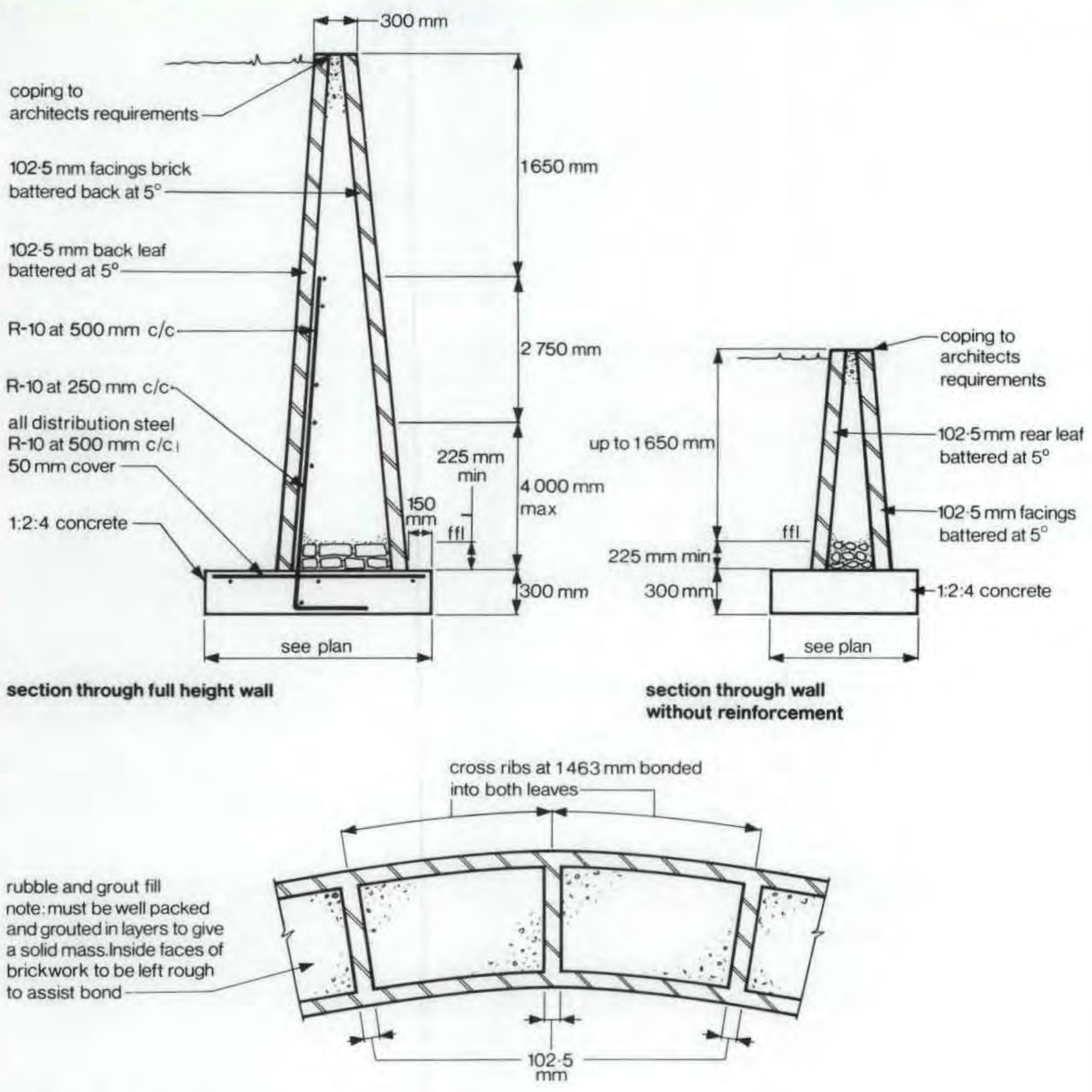


plan of section

Left and overleaf *Mass retaining walls in diaphragm construction (see Figs 9 and 10). Freedom Gardens, Ashton-under-Lyne. Architect: Alan Shaw. Structural engineers: W. G. Curtin & Partners.*

Above *Two 7.62 m high test walls, with an air bag between, in the UMIST laboratory. The walls were deliberately made too slender, with cross-ribs spaced too far apart, and were built to very average standards of construction in moderate strength bricks.*

subsidence due to coal extraction below ground. The stiff composite structure performed admirably in a situation where other forms of construction may have suffered considerable subsidence damage.

Whilst most of the applications of diaphragm walls have tended to concentrate on their effectiveness to resist lateral loading, they also possess ideal properties to resist axial loads applied at a great height, such as from high loading platforms, due to their ability to be designed to work at efficient stress levels and at the same time resist buckling.

RESEARCH AND DEVELOPMENT

The results of research work on half scale diaphragm walls (mainly financed by the BDA and BRE) have been published*.

The research provided ample evidence of the integral structural action of the cross-ribs and leaves in forming box sections of high lateral and vertical load resistance. Extrapolation of the results, to predict the lateral load resistance of full scale walls, has been recently confirmed in further research at UMIST Department of Civil and Structural Engineering.

This further research work, again mainly financed by the BDA, (and recently supported by both The Royal Society and The Science and Engineering Research Council, who have granted W. G. Curtin an Industrial Research Fellowship) is showing that the application of pre-stressing will lead to important developments for diaphragm walls in earth retaining structures, tanks, etc.

Because of the diaphragm wall's high $\frac{Z}{A}$ ratio and radius of gyration, it is obviously an ideal section for pre-stressing. It is hoped to publish the results of this current research work in 1982.

Two post-tensioned diaphragm wall projects have been built (one to resist in-plane tensile stresses resulting from differential settlement due to mining subsidence, and the other acting as a free cantilever) and are proving satisfactory in behaviour.

There is no doubt that future research work will increase the potential and widen the applications of the diaphragm wall technique.

* *Brick Diaphragm Walls – Research and Testing. Curtin & Sawko. The Structural Engineer 58B, No 1, March 1980.*

DESIGN PRINCIPLES

2

The design of earlier diaphragm walls was carried out on the basis of reasonable assumptions, some of which were then unproven.

Research work to date has confirmed the assumptions made in these early designs, permitting current and future designs to be made with even greater confidence. The basic design principle has been accepted by local authorities to whom calculations have been submitted for Building Regulation approval.

The design of a diaphragm wall is rarely governed by the compressive stresses in the brickwork, (as is the case in most brick structures) but by the wall's resistance to lateral forces due to wind. This consideration determines the spacing of the leaves. The centres of the ribs are usually governed by the need to transfer the shear stresses from the ribs to the leaves. The resulting compressive stresses in the brickwork tend to be very low, so that bricks of low compressive strength are usually structurally adequate.

The Code of Practice for Structural Masonry, BS 5628: Part 1 does not give adequate guidance on the design of complex masonry elements such as the diaphragm wall. This present guide has, therefore, been re-written in limit state terms, interpreting from BS 5628: Part 1 those principles which the authors consider to be relevant.

DESIGN SYMBOLS

Certain aspects of the design process in the worked examples which follow later will, of necessity, vary from the procedures given in BS 5628: Part 1 because the Code of Practice discusses plane wall sections only. As a result, it has been found necessary to introduce extra symbols, additional to those provided in BS 5628: Part 1 and, in order to avoid confusion, a full list of all the symbols used throughout the text and the worked examples is included with the extra symbols marked with an asterisk *.

A	area of masonry
*B	distance between centres of cross-ribs
*b	length of void between cross-ribs
*b_r	thickness of cross-rib
C_{pe}	external pressure coefficient (wind)
C_{pi}	internal pressure coefficient (wind)
*D	overall thickness of diaphragm wall
*d	width of void between flanges
e_x	eccentricity of loading at top of wall
f_k	characteristic compressive strength of masonry
f_{kx}	characteristic flexural strength (tension) of masonry
f_v	characteristic shear strength of masonry
*f_{ubc}	applied flexural compressive stress at design load
*f_{ubt}	applied flexural tensile stress at design load
G_k	characteristic dead load
g_d	design vertical dead load per unit area
h	clear height of wall or column between lateral supports
I	second moment of area

- *K_1 shear stress coefficient
- *K_2 stability moment trial section coefficient
- M applied bending moment
- *MR_s stability moment of resistance
- *M_w applied moment in height of wall
- *p_{ubc} allowable flexural compressive stress
- *p_{ubt} allowable flexural tensile stress
- Q_k characteristic imposed load
- q dynamic wind pressure
- SR slenderness ratio
- *t_f leaf (or flange) thickness
- V design shear force
- v_h design shear stress
- W_k characteristic wind load
- *w_s minimum width of stress block
- $\bar{y}$ dimension of centroid of section to centroid of stressed area
- Z section modulus
- β capacity reduction factor
- γ_f partial safety factor for loads
- γ_m partial safety factor for materials
- γ_{mv} partial safety factor for material in shear

VERTICAL LOADING

Slenderness ratio

Whilst in many cases of single storey structures vertical loading is not critical, it is considered sensible to relate the design of diaphragm walls to the requirements of the codes where possible. Thus it is necessary to assess the slenderness ratio of such walls, and to check that it does not exceed 27.

Effective height

There is a problem in determining the effective height of a diaphragm wall. If the wall is considered as a propped cantilever, it would be reasonable to suggest that the effective height is 0.75 times the actual height. However, under the action of wind pressure on the wall and suction on the roof, the value of the prop could be reduced and the effective height could be greater than 0.75 times the actual height. The assessment of the effective height must, therefore, be judged by the designer for each individual case.

Effective thickness

In BS 5628: Part 1, the slenderness ratio of a wall is defined as the ratio of effective height to effective thickness, because the code only takes account of solid plane wall sections, and for these the radius of gyration (used as the basis of considerations of slenderness in other structural materials) has a direct relationship with the thickness of the wall. In the code, walls with piers are treated as equivalent solid walls by the application of an adjusting factor.

For complex wall shapes such as the diaphragm wall, this approach is clearly inadequate and it is the authors view that the slenderness ratio for such walls, in fact all walls, should be based on radius of gyration.

Consider a solid wall section of unit length,

$$I = \frac{1 \times t_f^3}{12}$$

$$A = 1 \times t_f$$

Radius of gyration $r = \sqrt{\frac{I}{A}} = \sqrt{\frac{t_f^3}{12 t_f}} = \frac{1}{3.46} t_f$ ——— ①

To compare the thickness of a diaphragm wall with the thickness of a solid wall to give equivalent slenderness, take diaphragm section 10 from Table 1 (page 31) shown in figure 11.

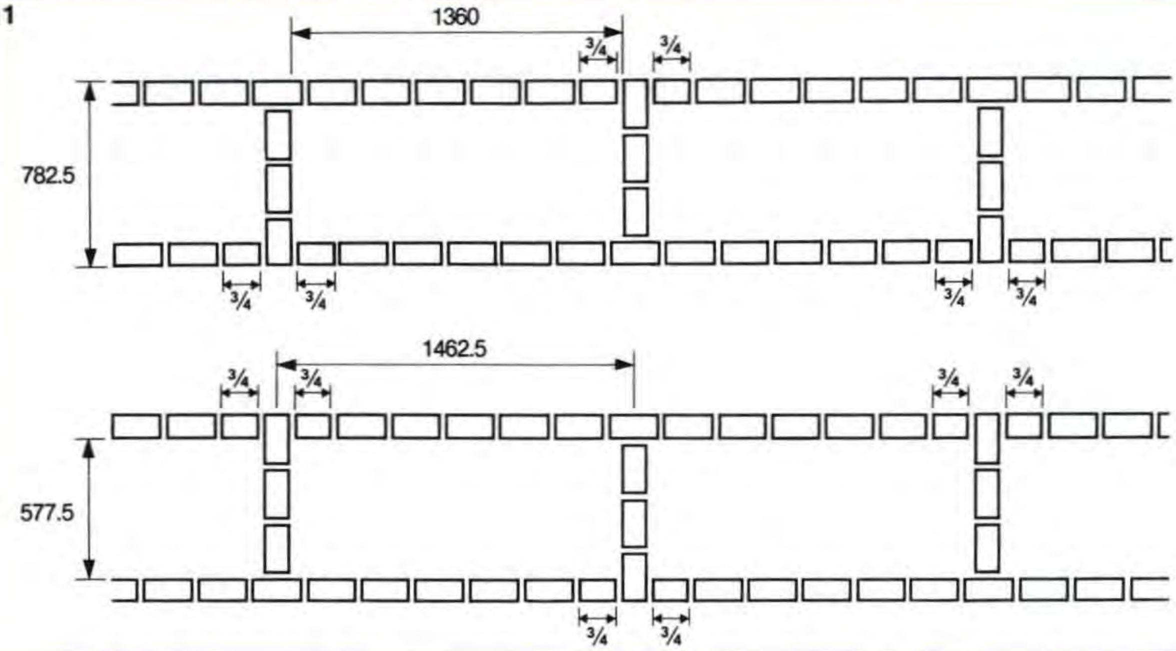
Radius of gyration $r = \sqrt{\frac{I}{A}} = \sqrt{\frac{24.99 \times 10^{-3}}{0.245}} = 0.32$ m ——— ②

$\therefore$ equivalent solid section requires thickness of

$$t_f = 0.32 \times 3.46 = 1.110 \text{ m}$$

Hence a diaphragm wall of 782.5 mm overall thickness has equivalent slenderness to a solid wall of 1.110 m thick. This, of course, does not imply an equal vertical load carrying capacity.

11

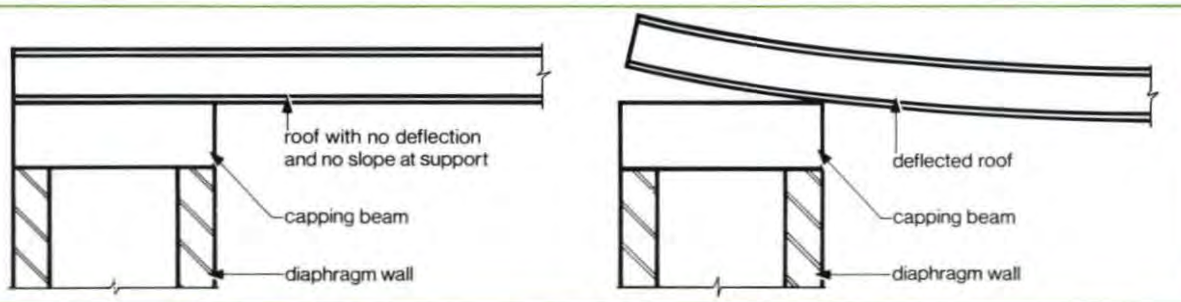


Although it may be argued that Table 7 in BS 5628: Part 1 can be easily modified to give slenderness ratios based on radius of gyration by multiplying the slenderness ratios by 3.46.

For the purposes of this design guide, the code table has been adopted on the assumption that the overall thickness of a diaphragm is taken as the effective thickness. This, of course, will give a very conservative design solution.

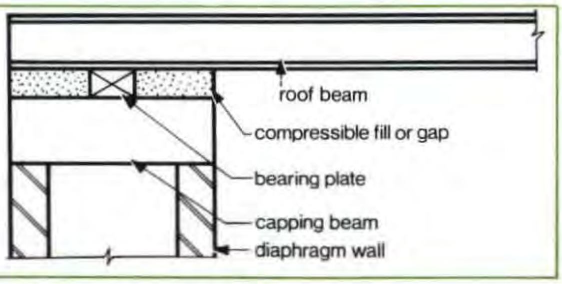
Eccentricity of vertical loading
A further problem arises in attempting to estimate the eccentricity of vertical loading. Most diaphragm walls designed by W. G. Curtin & Partners have been capped by reinforced concrete capping beams, to which the roofs have been bolted. If the roofs do not deflect, there would be zero slope of the roof members at their connection to the capping beam. However, such a theoretical condition does not arise for the roofs will deflect, even under their own weight alone. There is then a slope of the roof members at the support, resulting in the roof/wall contact not being concentric but eccentric. In the extreme case, the contact could be at the inner face of the inner leaf (figure 12).

12



Such an extreme case is hardly likely to occur in practice, since there will be some dispersal of the contact pressure through the capping beam. This disperson is, however, unlikely to be sufficient to cause the outer and inner leaves to be loaded equally. The inner leaf will still be more heavily stressed than the outer leaf. This, for bonded cross-ribs, could be considered as a local bearing stress, since the roof beam loads are applied at intervals and Clause 34 of BS 5628: Part 1 allows up to 50% increase in the local stress. The rib probably further disperses the excess stress in the inner leaf to the outer leaf. For tied cross-ribs, the designer should assess the effect of the eccentricity on an individual job basis. The problem of eccentricity can be controlled by the detailing of the bearing of the roof beams on the capping beams so that the load is applied where the designer wants it (figure 13).

13



Capacity reduction factor β
The capacity reduction factor, which allows for the effects of slenderness and eccentricity of loading in determining the design vertical load resistance of the wall, may now be obtained from table 7, BS 5628: Part 1.

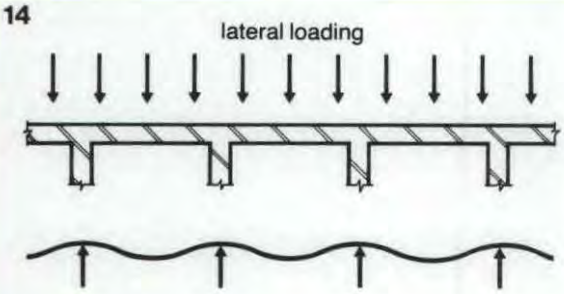
The designer should also consider the possibility of localised buckling of the leaves between the ribs, and/or the ribs between the leaves, in assessing the most onerous capacity reduction factor for design purposes.

LATERAL LOADING

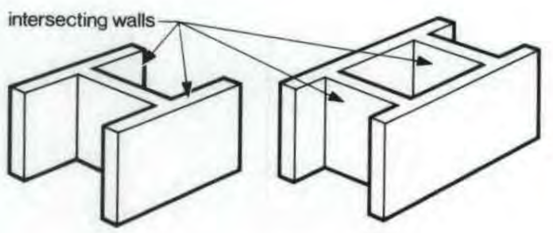
Determination of centres of cross-ribs
The centres of the cross-ribs are governed by the following conditions:
(a) The leaves acting as continuous horizontal slabs, subjected to wind loading, supported by and spanning between the cross-ribs (figure 14).

(b) As a wall subject to buckling under vertical loading. The effective length of the wall is taken as either the effective vertical height, or the length measured between adjacent intersecting walls, ie, the cross-ribs (figure 15).

14

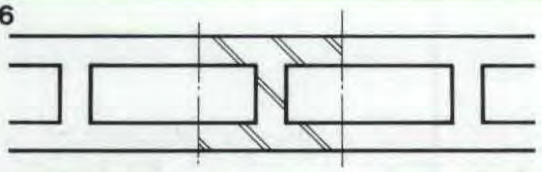


15



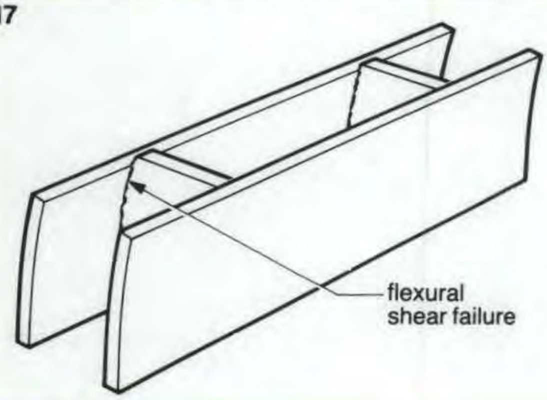
(c) Leaves and ribs acting together to form a series of I sections. The length of the flanges of the I should be restricted in accordance with Clause 36.4.3, BS 5628: Part 1 (figure 16) for the design of the effective section, although the centres of the cross-ribs may be greater.

16



(d) If the cross-ribs are spaced too widely, there may be flexural shear failure (in attempting to develop the box action of the section) between the ribs and the leaves, and particular attention should be given when designing metal ties (figure 17).

17



Calculating the cross-rib centres from these conditions gives:

Case (a)
$M \leqslant p_{ubt}Z$
where:

M $=$ applied bending moment due to wind $= \dfrac{\gamma_f W_k B^2}{10}$

p_{ubt} = allowable flexural tensile stress $\quad = \dfrac{f_{kx}}{\gamma_m}$

Z = section modulus $\quad = \dfrac{bt_f^2}{6}$

Example:

assuming, $W_k = 0.573$ kN/m²

leaf thickness $t_f = 0.1025$m

B = centres of cross-ribs

$f_{kx} = 0.9$ N/mm² (ie, clay bricks having a water absorption over 12% set in a designation (iii) mortar, see Table 3, BS 5628: Part 1)

$\gamma_m = 2.5$ (see Table 4, BS 5628: Part 1)

hence,

$$M = \frac{\gamma_f W_k B^2}{10} = \frac{1.4 \times 0.573 \times B^2}{10} = 0.08\ B^2\ \text{kNm}$$

$$p_{ubt} = \frac{f_{kx}}{\gamma_m} = \frac{0.9}{2.5} = 0.36\ \text{N/mm}^2$$

$$Z = \frac{bt_f^2}{6} = \frac{1 \times 0.1025^2}{6} = 1.75 \times 10^{-3}\ \text{m}^3$$

$$M = p_{ubt} Z$$

$$0.08B^2 = 0.36 \times 10^3 \times 1.75 \times 10^{-3}$$

$$B^2 = \frac{0.36 \times 1.75}{0.08}$$

$$B = \sqrt{\frac{0.36 \times 1.75}{0.08}} = 2.81\ \text{m centres of cross-ribs}$$

Case (b)

maximum slenderness ratio = 27 (BS 5628: Part 1, clause 28.1)

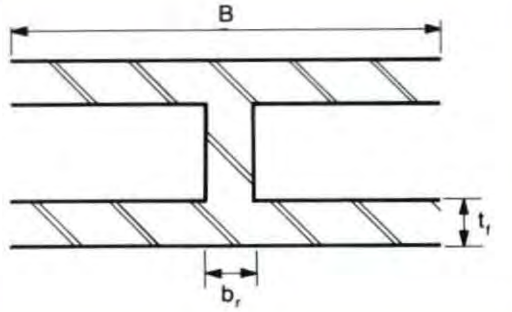
$$\frac{B}{t_f} = 27$$

$$B = 27 \times 0.1025 = 2.77\ \text{m centres of cross-ribs}$$

Case (c)

BS 5628: Part 1, clause 36.4.3 states that, in assessing the section modulus of a wall including piers, the outstanding length of the flange from the face of the pier should be taken as 6 × the thickness of the flange where the flange is continuous, but in no case more than the distance between the piers (ribs).

18



It is considered that the effective flange width should also be limited to a proportion of the height of the wall. As no such limitation is provided for in BS 5628: Part 1, it is proposed that ⅓ of the wall height, as was applicable in CP 114, 1969, clause 311 (e), would be an acceptable limit.

Example:

$B = 6t_f + 6t_f + b_r$ (figure 18)

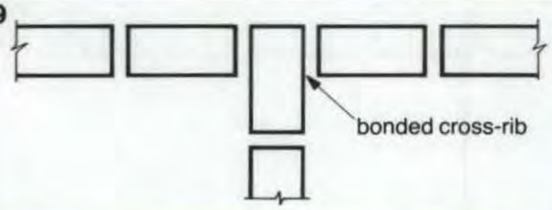
but, $t_f = b_r = 102.5$

hence $B = 13 \times 102.5 = 1.33$ m centres of cross-ribs

For the flange width to be restricted to one-third of the height of the wall, the wall in this example would require to be less than 4.0 m (3 × 1.33 m) in height for this criterion to be the limiting condition.

Case (d)
The shear resistance can be obtained either by bonding every alternate course of the cross-rib into the leaf (figure 19) or by using metal shear ties (figure 20).

19



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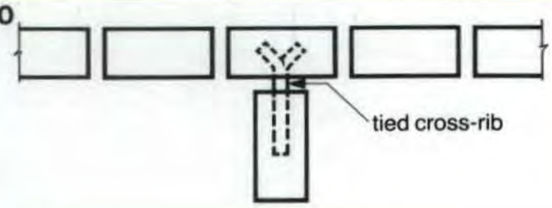
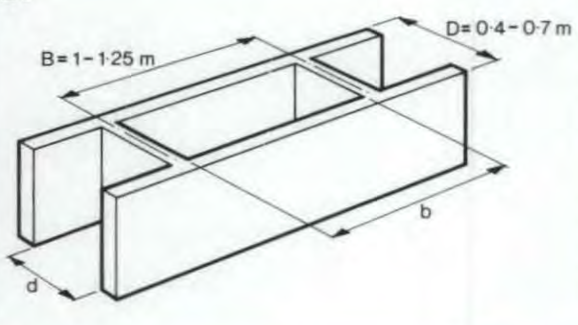


From experience, with wind forces of around 0.6 kN/(m^2) and wall heights of 8 m, it has been found that the rib spacings should be at about 1.0 m to 1.25 m centres. The rib centres calculated by method (c) generally dictate the design. However, the cross-ribs may be spaced further apart than this provided that only this length of flange is considered as resisting the bending. All other stress criteria must of course also be satisfied.

Depth of diaphragm wall
Obviously, the greater the depth of the wall, the greater is its resistance to wind forces. Increase in depth also improves the wall's slenderness ratio, and thus its axial loadbearing capacity. From experience with wind forces and wall heights mentioned above, the wall needs to be 0.4–0.7 m deep (figure 21).

21



Properties of section
Breadths and depths of diaphragm walls are governed mainly by brick sizes and joint thicknesses. The engineer is free to design the diaphragm best suited to his project. Figure 22 shows some typical breadths and depths, found useful in practice, based on the standard brick.

The calculations for a typical section are given below:

$$\text{B} = \underset{\text{(bricks)}}{[(4\times215)} + \underset{\text{(joints)}}{(5\times10)} + \underset{\text{(rib)}}{(2\times\tfrac{1}{2}\times102.5)]}\times10^{-3} \qquad = 1.0125\ \text{m}$$

$$\text{b} = 1.0125 - 0.1025 \qquad = 0.910\ \text{m}$$

$$\text{D} = [(2\times215)+10]\times10^{-3} \qquad = 0.440\ \text{m}$$

$$\text{d} = 0.440-(2\times0.1025) \qquad = 0.235\ \text{m}$$

$$\text{I} = \frac{\text{BD}^3}{12} - \frac{\text{bd}^3}{12} = \frac{1.0125\times0.44^3}{12} - \frac{0.91\times0.235^3}{12} \qquad = 6.21\times10^{-3}\ \text{m}^4$$

$$\text{Z} = \frac{\text{I}}{\text{y}} \qquad = \frac{6.21\times10^{-3}}{0.44\times0.5} \qquad = 28.23\times10^{-3}\ \text{m}^3$$

$$\text{A} = \text{BD}-\text{bd} \qquad = (1.0125\times0.44)-(0.91\times0.235) \qquad = 0.232\ \text{m}^2$$

The values per metre length of the wall are:

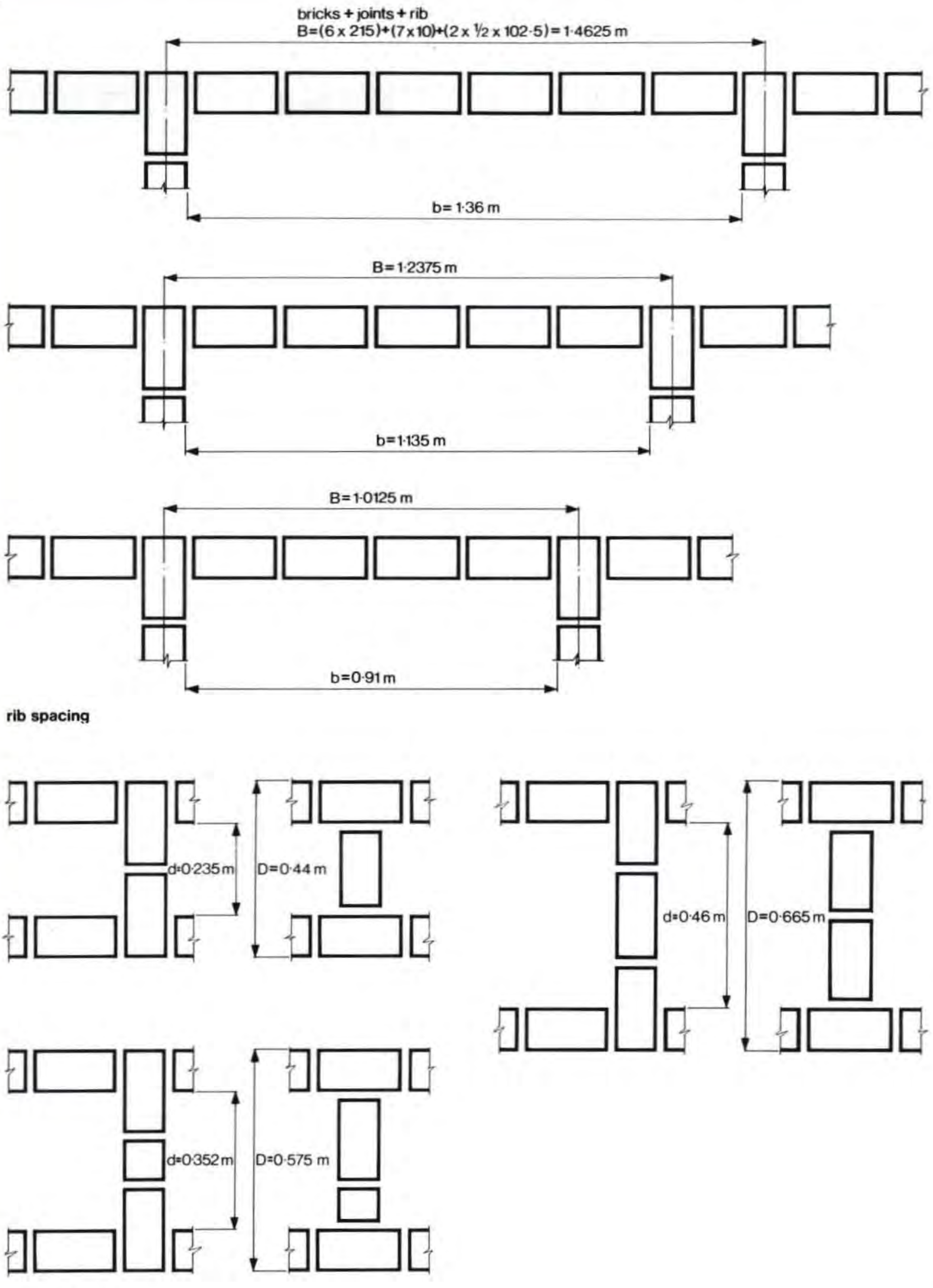
$$\text{I} = \frac{6.21}{1.0125} \qquad = 6.13\times10^{-3}\ \text{m}^4$$

$$\text{Z} = \frac{28.23}{1.0125} \qquad = 27.88\times10^{-3}\ \text{m}^3$$

$$\text{A} = \frac{0.232}{1.0125} \qquad = 0.229\ \text{m}^2$$

The section properties shown above, and others for a range of walls likely to be required, are given in Table 1 page 31.

22



common depths

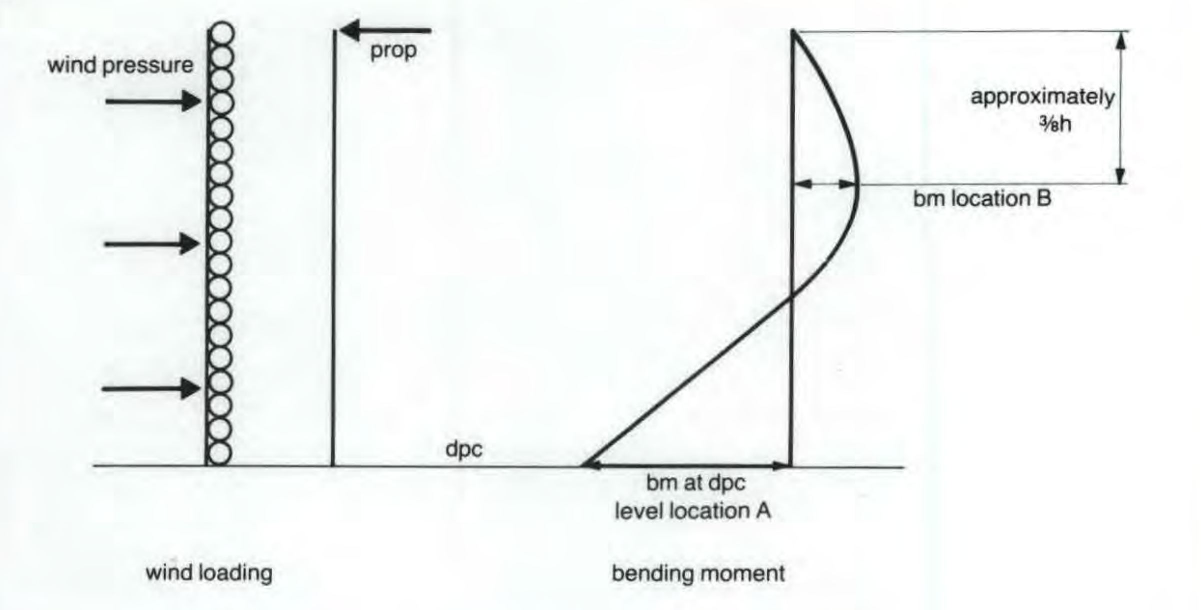
ASSUMED BEHAVIOUR OF A DIAPHRAGM WALL

For single-storey buildings, the critical design condition is rarely governed by axial compressive loading, but rather by lateral loading from wind forces. The limiting stresses are generally on the tensile face of the wall, and diaphragm walls are well equipped to resist these stresses. Furthermore, the roofs are generally designed and detailed to act as a prop or tie to the heads of the walls, and the design bending moment is taken to be similar to a propped cantilever.

Within the height of the wall there are two locations of critical bending moments:
level A = at the base of the wall, which is generally at dpc level, where a cracked section is assumed;

level B =at a level approximately $\frac{3}{8}$h down from the top of the wall where an uncracked section is assumed (figure 23).

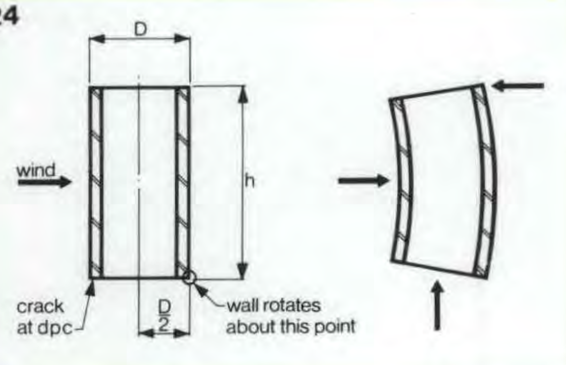
23



The resistance to these two levels of critical bending moment is provided by:
at level A =the stability moment of resistance (MR_s) of the cracked wall and
at level B =the flexural tensile resistance of the wall.

Stability moment of resistance (MR_s)
Single-storey buildings tend to have light-weight roof construction and low superimposed roof loading. Hence, the forces and moments due to lateral wind pressure have greater effect on the stresses in the supporting masonry than they do in multi-storey buildings. Since there is little precompression, the wall's stability relies more on its own gravitational mass (including any nett roof loads) and the resulting resistance moment. Under lateral wind pressure loading, the wall will tend to rotate at dpc level on its leeward face and 'crack' at the same level on the windward face as indicated in figure 24.

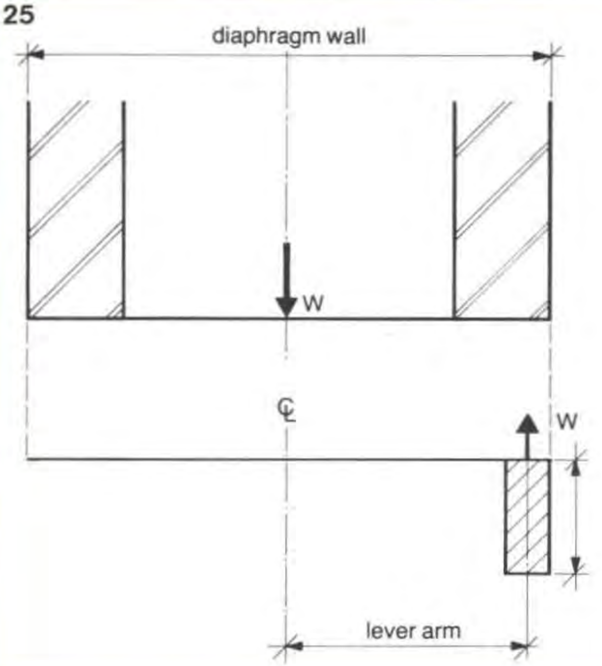
24



In limit state design, the previous knife-edge concept of the point of rotation is replaced with a rectangular stressed area, in which the minimum width of masonry is stressed to ultimate to produce the maximum lever arm for the axial load to generate the maximum stability moment of resistance MR_s, as indicated in figure 25.

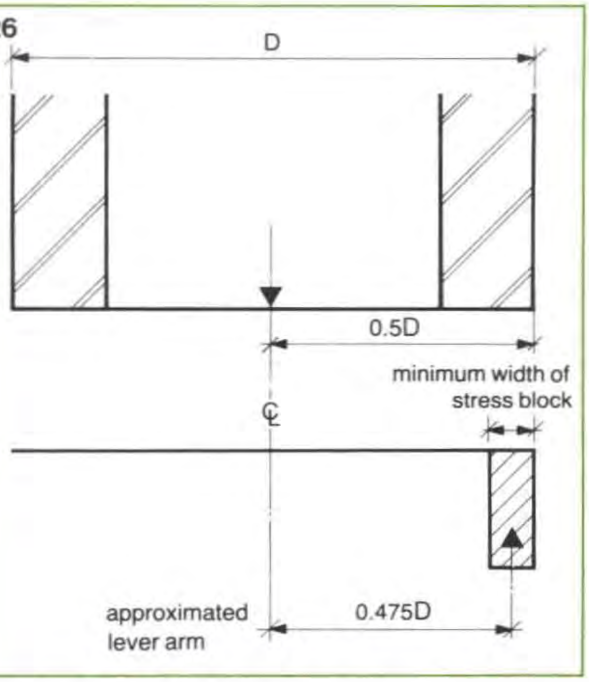
For the purpose only of assessing a trial section, this lever arm is approximated to 0.475 D as shown in figure 26 and discussed later under 'Trial Section Coefficients K_2 and Z' (page 29).

25

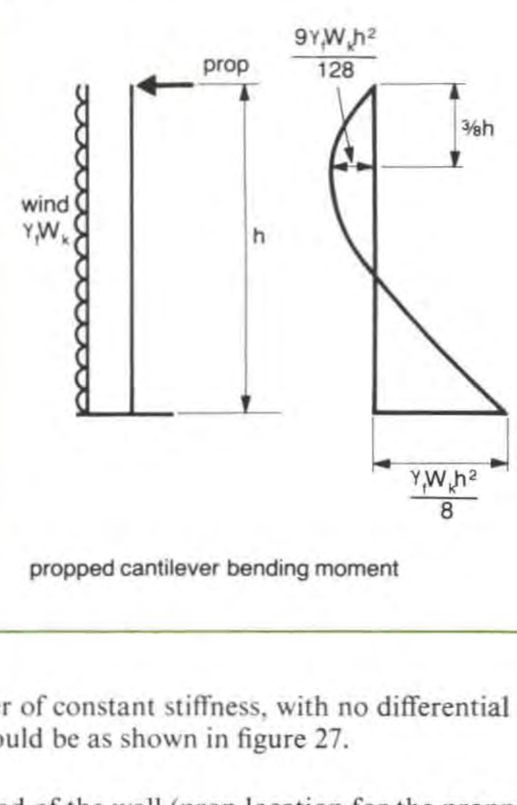


minimum width of stress block is stressed to ultimate to provide the maximum lever arm for the axial load W to generate the maximum stability moment of resistance MR_s

26



27



propped cantilever bending moment

Consider levels of critical stress

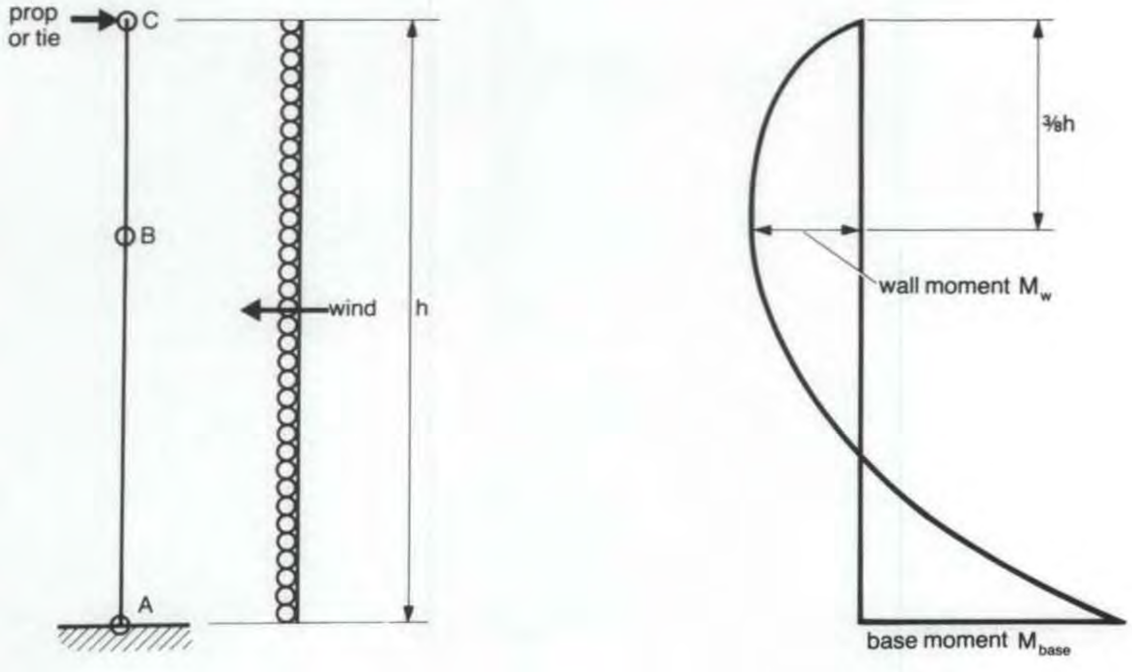
For a uniformly distributed load on a propped cantilever of constant stiffness, with no differential movement of the prop, the bending moment diagram would be as shown in figure 27.

However, in reality, some deflection will occur at the head of the wall (prop location for the propped cantilever design), and the wall is not of constant stiffness throughout its height due to changes in the effective section at crack positions. It is, therefore, a coincidence if the resistance moment at the base is exactly equal to $\frac{\gamma_f W_k h^2}{8}$ which is applicable to a true propped cantilever. Thus, it is usually necessary to adjust the bending moment diagram from that of a true propped cantilever, as will be explained later. The upper level of critical stress does not necessarily occur at $\frac{3}{8}h$ from the top of the wall but should be calculated to coincide with the point of zero shear on the adjusted bending moment diagram. The second level of critical stress to be considered will still occur at the base of the wall, and is resisted by the stability moment of resistance. It is considered unwise to include, as contributing to the stability moment of resistance, any flexural tensile strength which the dpc may be claimed to possess. The explanation for this involves the application of a 'plastic' analysis to the failure mechanism of the wall.

'Plastic' analysis (crack hinge analysis)
The 'plastic' analysis of the wall action considers the development of 'plastic' hinges (or 'crack' hinges) and the implications of the mechanisms of failure.

Referring to figure 28:

28



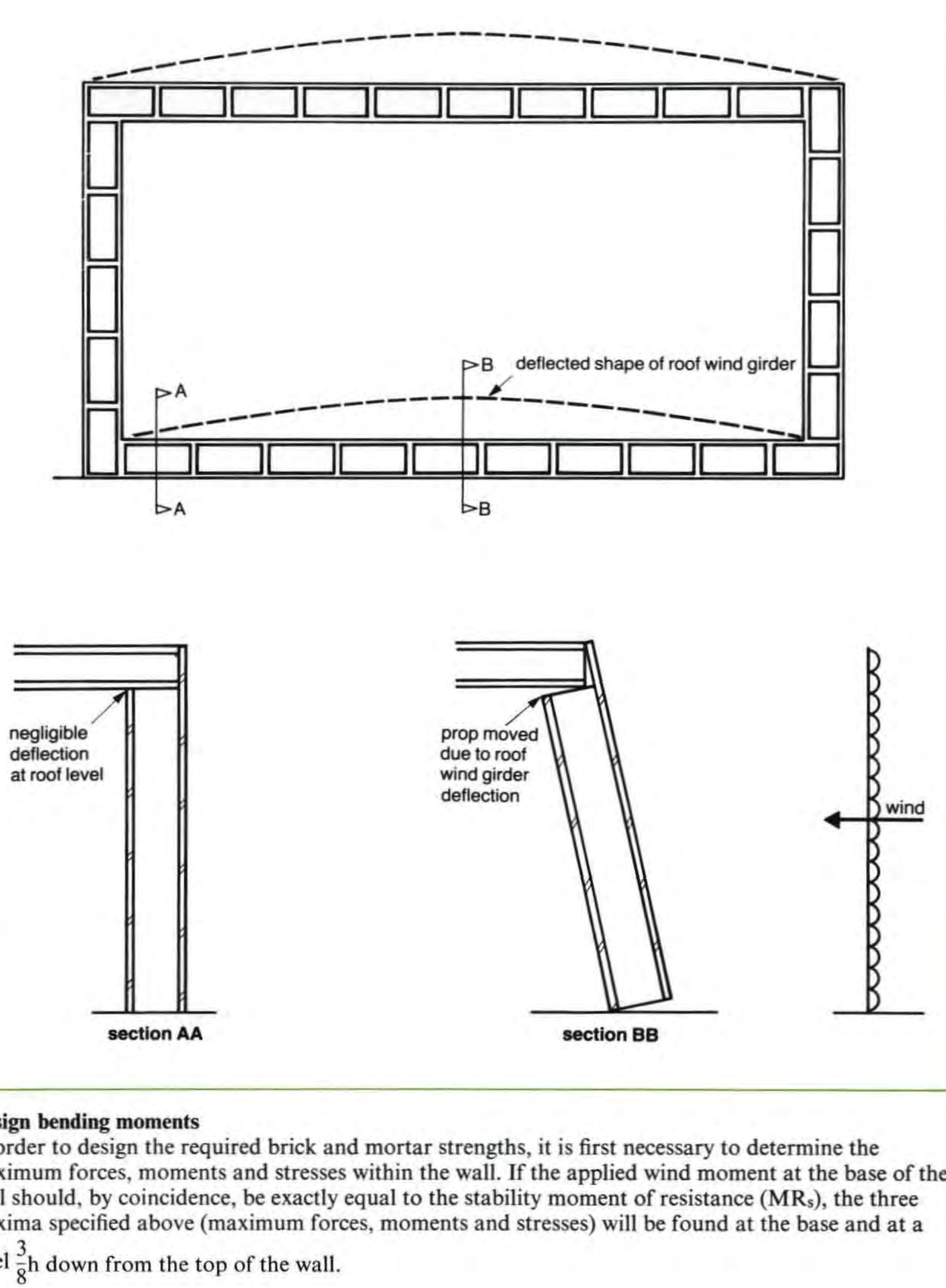
Three 'plastic' hinges are necessary to produce failure of the propped cantilever shown, and these will occur at locations A, B and C. Location C, the prop, is taken to be a permanent hinge. Hence, under lateral loading, the two hinges at A and B require full analysis.

As the lateral loading is applied the wall will flex, moments will develop to a maximum at A and B, and the roof plate action will provide the propping force at C.

As the roof plate is unlikely to be absolutely rigid, some deflection must be considered to occur which will allow the prop at the head of the wall to move and the wall as a whole to rotate. This deflection of the roof plate will be a maximum at midspan and zero at the gable shear wall positions – see figure 29.

Thus, each individual cross-rib will be subjected to slightly differing loading/rotation conditions. If, in addition to the stability moment of resistance at base level, flexural tensile resistance is also exploited to increase the resistance moment, there is a considerable danger that rotation, due to the deflection of the roof plate prop, may eliminate this flexural tensile resistance by causing the wall to crack at base level. The effect of this additional rotation would be an instantaneous reduction in resistance moment at this level. This, in turn, would require the wall section at level B to resist the excess loading transferred to that level, and this could well exceed the resistance moment available at that level. Hence, the two 'plastic' hinges at levels A and B could occur simultaneously, giving failure at a loading less than that calculated on the basis of tensile resistance at the base. If, however, the flexural tensile resistance at the base is ignored, the design bending moment diagram will utilise only the stability moment of resistance at base level, and this will remain unaffected by whatever rotation may occur due to the deflection of the roof prop.

29



Design bending moments

In order to design the required brick and mortar strengths, it is first necessary to determine the maximum forces, moments and stresses within the wall. If the applied wind moment at the base of the wall should, by coincidence, be exactly equal to the stability moment of resistance (MR_s), the three maxima specified above (maximum forces, moments and stresses) will be found at the base and at a level $\frac{3}{8}$h down from the top of the wall.

If the MR_s is less than the applied base wind moment of $\frac{\gamma_f W_k h^2}{8}$, or if significant lateral deflection of the roof prop occurs, the wall will tend to rotate and 'crack' at the base. Providing that no tensile resistance exists at this level, the MR_s will not decrease because the small rotation will cause an insignificant reduction in the lever arm of the vertical load. However, on the adjusted bending moment diagram, the level of the maximum wall moment will not now be at $\frac{3}{8}$h down from the top and its value will exceed $\frac{9\gamma_f W_k h^2}{128}$.

For example, suppose the numerical value of a particular MR_s is equivalent to, say $\frac{\gamma_f W_k h^2}{10}$, then the reactions at base and prop levels would be:

$=\frac{\gamma_f W_k h}{2} \pm \frac{\gamma_f W_k h^2}{10h}$

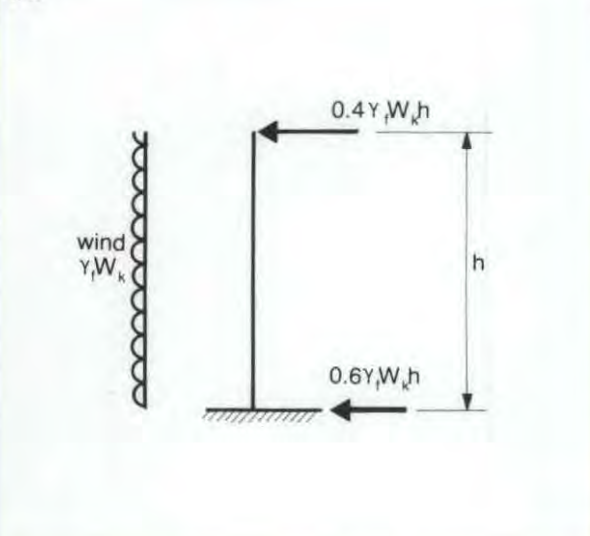
$=0.5\ \gamma_f W_k h \pm 0.1\ \gamma_f W_k h$

$=0.6\ \gamma_f W_k h$ at base level

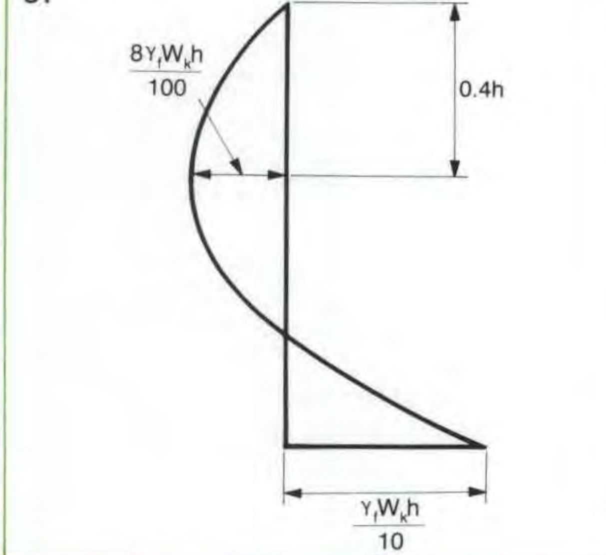
$=0.4\ \gamma_f W_k h$ at prop level (see figure 30)

The MR_s is inadequate to resist a true propped cantilever base moment of $\frac{\gamma_f W_k h^2}{8}$. Hence, the section will crack, and any additional load resistance available at the higher level will come into play. The true propped cantilever BM diagram is adjusted to allow a greater share of the total load resistance to be provided by the stiffness of the wall within its height, and the adjusted BM diagram for the example under consideration is shown in figure 31.

30



31



The applied wind moment at the level 0.4 h down is calculated as:
$(0.4\ \gamma_f W_k h \times 0.4\ h) - (0.4\ \gamma_f W_k h \times 0.2\ h) = 0.08\ \gamma_f W_k h^2$ which exceeds the true propped cantilever wall moment of $\frac{9\ \gamma_f W_k h^2}{128}$ $(0.07\ \gamma_f W_k h^2)$. The moment of resistance provided by the wall at this level must then be checked against the calculated maximum design bending moment.

The action of the wall is perhaps better described as that of a member simply supported at prop level, and partially fixed at base level where the partial fixity can be as high as $\frac{\gamma_f W_k h^2}{8}$, that of a true propped cantilever.

A rigid prop is not possible in practice (nor is a perfectly 'pinned' joint or 'fully fixed-ended' strut, etc), but the initial assumption of a perfectly rigid prop generally provides the most onerous design condition. Considering the two locations of maximum design bending moments and development of the respective moments of resistance, it is apparent that the critical design condition invariably occurs at the higher location where the resistance is dependent on the development of both flexural compressive and flexural tensile stresses.

Allowable flexural stresses

(i) allowable flexural tensile stress, f_{ubt}

BS 5628: Part 1, clause 36.4.3 gives:

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

which is basically a stress times section modulus relationship, in which the stress $=\frac{f_{kx}}{\gamma_m}$

which, for the purposes of this design guide, we have termed f_{ubt}, allowable flexural tensile stress hence:

allowable flexural tensile stress, $f_{ubt}=\frac{f_{kx}}{\gamma_m}$

where f_{kx} = characteristic flexural strength (clause 24, BS 5628: Part 1),
and γ_m = partial safety factor for materials (clause 27, BS 5628: Part 1).

(ii) allowable flexural compressive stress, f_{ubc}

BS 5628: Part 1 gives no consideration to flexural compressive stresses in designing laterally loaded elements. In the derivation of β, in appendix B, the code discusses the application of a rectangular stress block of $\frac{1.1f_k}{\gamma_m}$ to the resistance of bending moments emanating from eccentric vertical loading.

Consideration must also be given to the implication of the geometric form of the diaphragm wall on the flexural compressive stress, where the whole of the width of the flange (or leaf) of the wall may be subject to the stress. In this situation, the possibility of local buckling of the flange must be allowed for in the assessment of the allowable flexural compressive stress, and the stress formula is written as $\frac{1.1\beta f_k}{\gamma_m}$ where β is the capacity reduction factor in respect of the local buckling condition.

Hence, allowable flexural compressive stress, $f_{ubc} = \frac{1.1\beta f_k}{\gamma_m}$.

Trial section coefficients K_2 and Z

The symmetrical profile of the diaphragm wall has permitted the development of a direct route to a trial section which considers the two critical conditions that exist in the 'propped cantilever' action of the analysis.

Condition (i) exists at the base of the wall where the applied bending moment at this level must not exceed the stability moment of resistance of the wall.

Condition (ii) exists at approximately $\frac{3}{8}$h down from the top of the wall where the flexural tensile stresses are a maximum and must not exceed those allowable through calculation.

Consider the two conditions

Condition (i)

The trial section analysis is simplified by assuming that the dpc at the base level does not transfer tensile forces and that the mass contributing to the MR_s comprises only the own weight of the masonry.

BM at base level $= \frac{\gamma_f W_k h^2}{8}$ ——— ③

MR_s at base level = Area × height × density × γ_f × 0.475 D (see Stability moment of resistance)

MR_s = 0.475 ($Ah\gamma_f D$ density) ——— ④

Equating ③ and ④,

$$\frac{\gamma_f W_k h^2}{8} \leqslant 0.475\,(Ah\gamma_f D \text{ density})$$

γ_f for wind and dead loads will be taken as 1.4 and 0.9 respectively.

Hence $0.175\ W_k h^2 \leqslant 0.4275$ (AhD density)

now let $K_2 = 0.4275$ AD density

then $W_k h \leqslant 5.714K_2$

$$h \leqslant \frac{5.714K_2}{W_k} \qquad ⑤$$

Condition (ii)

The trial section analysis is further simplified by assuming that flexural tensile stresses control, γ_m is taken as 2.5 and f_{kx} as 0.4 N/mm².

BM at $\frac{3}{8}$h level $= \frac{9\gamma_f W_k h^2}{128}$ ——— ⑥

moment of resistance $= \left(\frac{f_{kx}}{\gamma_m} + g_d\right)Z$ ——— ⑦

Equating ⑥ and ⑦

$$\frac{9\gamma_f W_k h^2}{128} \leqslant \left(\frac{f_{kx}}{\gamma_m} + g_d\right)Z$$

$$\frac{9 \times 1.4 \times W_k \times h^2}{128} \leqslant \left(\frac{0.4 \times 10^3}{2.5} + \frac{\gamma_f \times 20 \times 3 \times h}{8}\right)Z$$

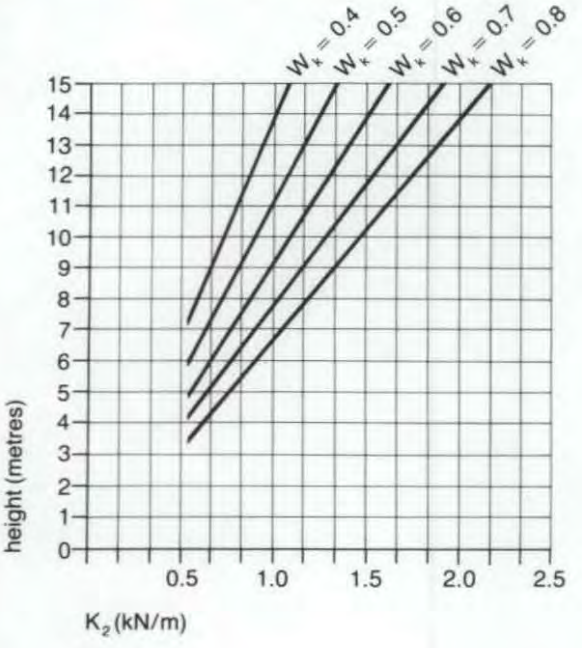
$$0.098\ W_k h^2 \leqslant (160 + 6.75\ h)Z$$

$$Z = \frac{0.098\ W_k h^2}{160 + 6.75\ h}$$

$$Z = \frac{W_k h^2}{1600 + 67.5\ h} \qquad ⑧$$

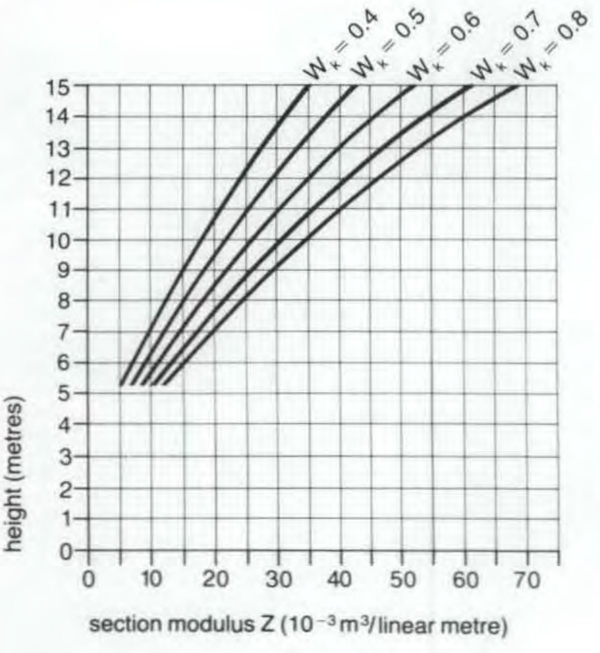
Two graphs have been plotted for equations ⑤ and ⑧ and for various values of W_k, which are shown in figures 32 and 33.

32



note: this trial section graph is based on the loading combination of dead plus wind for which the partial safety factors on loads (γ_f) are taken as 0.9 and 1.4 respectively.

33



note: this trial section graph is based on the loading combination of dead plus wind for which the partial safety factors on loads (γ_f) are taken as 0.9 and 1.4 respectively.

Then, for a known wall height and wind pressure, values of K_2 and Z may be read off the graphs and, using Table 1, the most suitable section can be obtained for full analysis. It should be remembered that the two trial section graphs have been drawn assuming fixed conditions for a number of variable quantities which are summarised thus:

(a) wall acts as a true propped cantilever

(b) dpc at base of wall cannot transfer tension

(c) vertical roof loads (downward or uplift) are ignored

(d) γ_m is taken to be 2.5

(e) f_{kx} is taken to be 0.4 N/mm^2

(f) density of masonry is taken to be 20 kN/m^3

(g) K_2 values calculated using approximated lever arm method.

The trial section graphs should only be used for the purpose of obtaining a trial section, and a full analysis of the selected section should always be carried out.

Table 1. Section properties

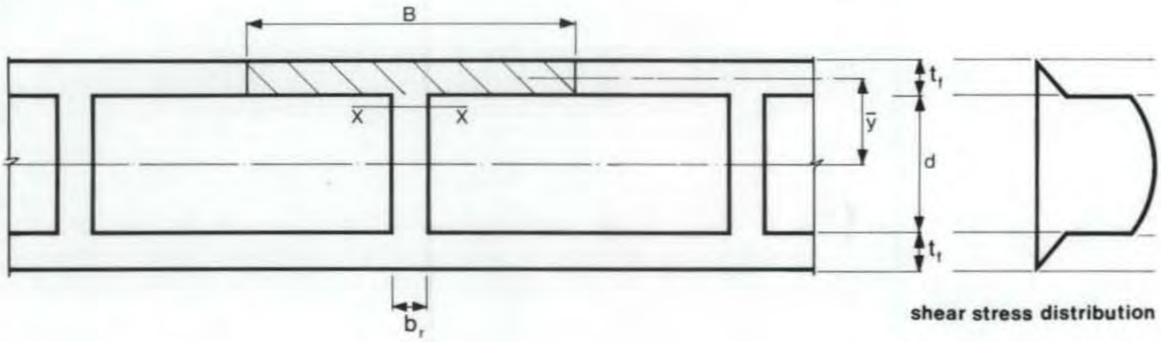
	Dimensions (in metres)				Section properties/diaphragm			Section properties/metre			Shear stress coefficient	Stability moment coefficient K_2 (kN/m) density = 20kN/m^3	K_2 when density = 18kN/m^3
Section	D	d	B	b	$I \times 10^{-3}m^4$	$Z \times 10^{-3}m^3$	A m^2	$I \times 10^{-3}m^4$	$Z \times 10^{-3}m^3$	A m^2	K_1/m^2		
1	0.44	0.235	1.4625	1.36	8.91	40.49	0.324	6.09	27.69	0.222	27.74	0.835	0.752
2	0.44	0.235	1.2375	1.135	7.55	34.32	0.278	6.10	27.73	0.225	27.66	0.846	0.762
3	0.44	0.235	1.0125	0.91	6.21	28.83	0.232	6.13	27.88	0.229	27.51	0.862	0.776
4	0.5575	0.352	1.4625	1.36	16.18	58.04	0.337	11.06	39.69	0.230	20.52	1.097	0.987
5	0.5575	0.352	1.2375	1.135	13.74	49.29	0.290	11.10	39.83	0.234	20.44	1.116	1.004
6	0.5575	0.352	1.0125	0.91	11.31	40.57	0.244	11.17	40.07	0.241	20.34	1.149	1.034
7	0.665	0.46	1.4625	1.36	24.81	74.62	0.347	16.96	51.02	0.237	16.56	1.348	1.212
8	0.665	0.46	1.2375	1.135	21.12	63.52	0.301	17.07	51.33	0.243	16.46	1.382	1.243
9	0.665	0.46	1.0125	0.91	17.43	52.43	0.254	17.21	51.77	0.251	16.37	1.427	1.284
10	0.7825	0.5775	1.4625	1.36	36.56	93.45	0.359	24.99	63.90	0.245	13.60	1.639	1.478
11	0.7825	0.5775	1.2375	1.135	31.18	79.69	0.313	25.19	64.40	0.253	13.49	1.693	1.523
12	0.7825	0.5775	1.0125	0.91	25.82	66.01	0.267	25.50	65.20	0.264	13.33	1.766	1.590
13	0.89	0.685	1.4625	1.36	49.46	111.14	0.37	33.82	76.00	0.253	11.64	1.925	1.733
14	0.89	0.685	1.2375	1.135	42.4	95.3	0.324	34.26	77.01	0.262	11.49	1.994	1.794
15	0.89	0.685	1.0125	0.91	34.86	78.34	0.278	34.43	77.37	0.274	11.44	2.085	1.877

Note: *For Sections 1, 4, 7, 10, 13 the flange length slightly exceeds the limitations given in Clause 36.4. 3 (b) BS 5628. These sections have been included since they are the closest brick sizes to the flanges recommended in the code. If the designer is concerned at this marginal variation he may calculate the section properties on the basis of an effective flange width of 1.33m.*

Shear stress coefficient K_1

It is necessary to check the shear stress at the junction of the cross-ribs and the leaves (figure 34).

34



Vertical design shear stress, $v_h = \dfrac{VA\bar{y}}{Ib_r}$ where V = design shear force

$= \gamma_f \times$ characteristic shear force

when $A = B \times t_f$ and $\bar{y} = \dfrac{d}{2} + \dfrac{t_f}{2}$ then at XX

vertical design shear stress, $v_h = \dfrac{V \times B \times t_f\left(\dfrac{d}{2} + \dfrac{t_f}{2}\right)}{I\,b_r} = \left(\dfrac{f_v}{\gamma_{mv}}\right)$

generally $t_f = b_r = 0.1025$ m

therefore $v_h = V \times \dfrac{B}{I}\left(\dfrac{d}{2} + \dfrac{0.1025}{2}\right)$

then $v_n = V\,K_1$

where $K_1 = \dfrac{B}{I}\left(\dfrac{d}{2} + \dfrac{0.1025}{2}\right)$ = shear stress coefficient

values of K_1 may be calculated for all diaphragm wall profiles and some are given in table 1.

Example

for wall section 3 (table 1)

$$K_1 = \frac{1.0125}{6.21 \times 10^{-3}}\left(\frac{0.235}{2} + \frac{0.1025}{2}\right)$$

$K_1 = 27.51$ per m^2

DESIGN PROCEDURE

3

A suggested design procedure is as follows:

1. Calculate characteristic and design loads from dead, superimposed and wind loading on building.

2. Select trial section.

3. Calculate ring beam size.

4. Assume brick and mortar strength/designation

5. Calculate applied wind moment at base of wall $\frac{\gamma_f W_k h^2}{8}$ and stability moment of resistance at base MR_s and compare.

6. Calculate position and magnitude of maximum applied wall moment M_w.

7. Calculate design flexural stresses at maximum M_w position.

8. Calculate flexural resistance of masonry at maximum M_w level and compare with design stresses.

9. Calculate shear stress in cross-ribs.

10. Design shear ties or check shear resistance of bonded cross-ribs.

11. Design roof plate and transverse shear walls (both of these latter design aspects are outside the scope of this design guide).

12. Check dead plus imposed plus wind loading combination.

13. Check dead plus imposed loading combination.

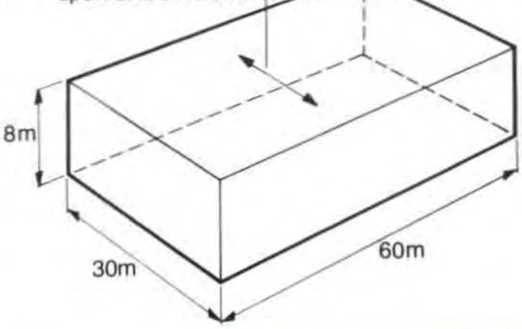
WORKED EXAMPLE NO. 1

Warehouse building

A warehouse measuring 30 m × 60 m and 8 m high is shown in figure 35 and is to be designed in brickwork, using diaphragm wall construction for its main vertical structure. There are no substantial internal walls within the building to provide any intermediate support. During construction, extensive testing of materials and strict supervision of workmanship will be employed.

Facing bricks with a compressive strength of 41.5 N/mm² and a water absorption of 8% will be used throughout the building, and are assumed to have a density of 20 kN/m³.

35



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.

In this example, these values are assumed to have been computed as:
Dynamic wind pressure, q $= 0.71$ kN/m^2
Walls:
C_{pe} on windward face $= 0.8$
C_{pe} on leeward face $= -0.5$
C_{pi} on walls, either $= +0.2$ or -0.3
Roof:
Gross wind uplift $= C_{pe} + C_{pi} = 0.60$
Therefore, characteristic wind loads are:
Pressure on windward wall $= W_{k1} = (C_{pe} - C_{pi})q = (0.8 + 0.3)0.71$
$= 0.781$ kN/m^2
Suction on leeward wall $= W_{k2} = (C_{pe} - C_{pi})q = (0.5 + 0.2)0.71$
$= 0.497$ kN/m^2
Gross roof uplift $= W_{k3} = (C_{pe} + C_{pi})q = 0.6 \times 0.71$
$= 0.426$ kN/m^2

(b) Dead and imposed loads
(i) Characteristic imposed load, Q_k, $= 0.75$ 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 kN/m^2
felt and chippings	=	0.30 kN/m^2
o.w. roof beams	=	0.19 kN/m^2
Total G_k	=	0.67 kN/m^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 + imposed + wind should be checked.
Design dead load $= 0.9\ G_k$ or $1.4\ G_k$
Design wind load $= 1.4\ W_k$ or $0.015\ G_k$

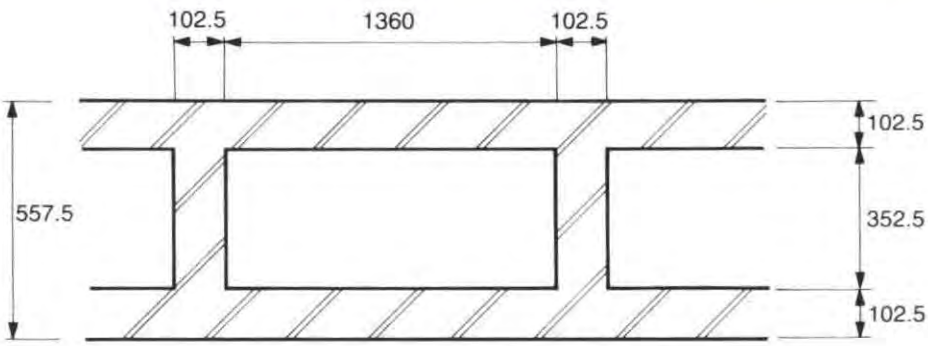
Therefore, by inspection, the most critical combinations of loading will be given by:
Design dead load $= 0.9 \times 0.67 = 0.603$ kN/m^2
Design wind loads:
Pressure, from $W_{k1} = 1.4 \times 0.781 = 1.093$ kN/m^2
Suction, from $W_{k2} = 1.4 \times 0.497 = 0.696$ kN/m^2
Uplift, from $W_{k3} = 1.4 \times 0.426 = 0.597$ kN/m^2
Design dead-uplift $= 0.603 - 0.597 = 0.006$ kN/m^2, say = zero.

2. Trial section
For the wall height of 8.0 m and the characteristic wind load of 0.781 kN/m^2,
$K_2 = 1.16$ kN/m and $Z = 23.3 \times 10^3$ m^3 can be read from figures 32 and 33 respectively.
Select wall section 4 (Table 1, page 31) and a full analysis using this section should then be carried out.

Wall properties
$I/m = 11.06 \times 10^{-3}$ m^4
$Z/m = 39.69 \times 10^{-3}$ m^3
$A/m = 0.230$ m^2
$K_1 = 20.52/\text{m}^2$

The wall section is shown in figure 36.

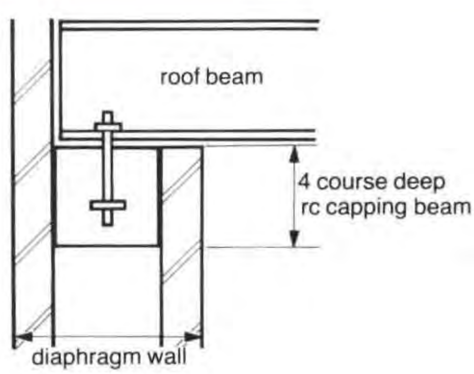
36



section4 from table 1

3. RC capping beams
The design wind uplift has been shown to be equal to the design dead load of the roof. However, to to provide an adequate factor of safety against roof uplift under abnormal wind forces, the four-course deep roof capping beam shown in figure 37 will be provided. This beam section will also provide a substantial anchorage for the main roof beams.

37



4. Brick and mortar specification
The facing bricks to be used throughout have been specified as having a compressive strength of 41.5 N/mm_2 and a water absorption of 8% (density 20 kN/m^3), and are to be set in a 1:1:6 designation (iii) mortar.

5. Determine wind moment and MR_s at base of wall
Consider 1 m length of wall:

design wind moment at base $= \dfrac{\gamma_f W_k h^2}{8}$

$$= \frac{1.4 \times 0.781 \times 8^2}{8}$$

$$= 8.744 \text{ kNm}$$

Stability moment of resistance = axial load × lever arm

The axial load for this example consists of the design dead load of the masonry only (as the wind uplift negates the roof dead loading) and is calculated as:
$\gamma_f \times \text{area} \times \text{density} \times \text{height} = 0.9 \times 0.230 \times 20 \times 8 = 33.12$ kN/m

In order to calculate the lever arm of the axial load (see figure 25) it is first necessary to establish the minimum width of the stress block.

minimum stress block width $w_s = \dfrac{\text{axial load}}{p_{ubc}}$

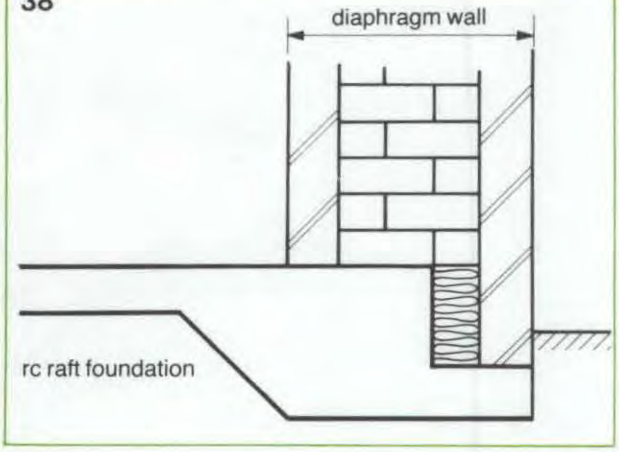
where p_{ubc} = allowable flexural compressive stress

$$= \frac{1.1\beta f_k}{\gamma_m}$$

At foundation level, where the foundation is assumed to comprise an rc raft as shown in figure 38, full restraint of the wall, against buckling, may be assumed, hence $\beta = 1.0$.

therefore $p_{ubc} = \dfrac{1.1 f_k}{\gamma_m}$

38



Facing bricks with a compressive strength of 41.5 N/mm² set in a 1:1:6 mortar, designation (iii), have been specified and therefore, from table 2a, BS 5628: Part 1 by interpolation, $f_k = 9.41$ N/mm², and with extensive materials testing and strict workmanship supervision, clause 27, BS 5628: Part 1 permits a value of 2.5 for γ_m, partial factor of safety for material strength.

Hence, $p_{ubc} = \dfrac{1.1 \times 9.41}{2.5} = 4.14$ N/mm²

therefore, min. width of stress block $w_s = \dfrac{33.12 \times 10^3}{1000 \times 4.14}$

(assuming the stress block to be within the solid portion of the wall)

therefore $w_s = 8.0$ mm

(ie, assumption correct).

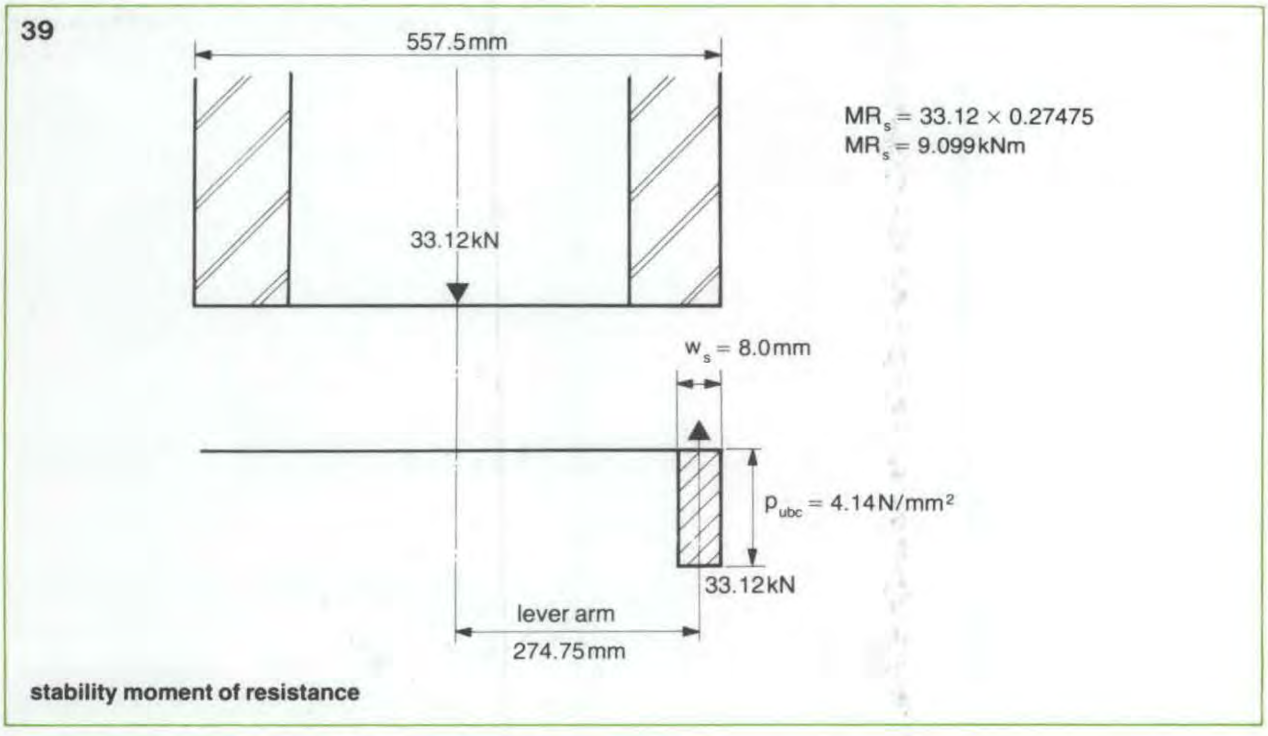
Therefore, lever arm $= \dfrac{\text{wall thickness}}{2} - \dfrac{w_s}{2} = 274.75$ mm

and stability moment of resistance, $MR_s = 33.12 \times 0.27475$

$MR_s = 9.099$ kNm (see figure 39)

Which is greater than the applied design wind moment (at the base) of 8.744 kNm.

39



stability moment of resistance

6. Maximum wind moment M_w in span of wall

Since the stability moment of resistance at the base exceeds the applied design wind moment, the wall is assumed to act as a true 'propped cantilever' and the maximum applied design wind moment in the span is, therefore, located at $\frac{3}{8}$h down from the roof prop.

Therefore, design wind moment at $\frac{3}{8}$ h, $M_w = \frac{9\gamma_f W_k h^2}{128}$

$$= \frac{9 \times 1.4 \times 0.781 \times 8^2}{128}$$

$$= 4.92 \text{ kNm}$$

The design bending moment diagram can now be drawn as shown in figure 40.

It is interesting to compare the calculated value of the stability moment of resistance, MR_s, with the approximate lever arm method suggested earlier for the calculation of the trial section.

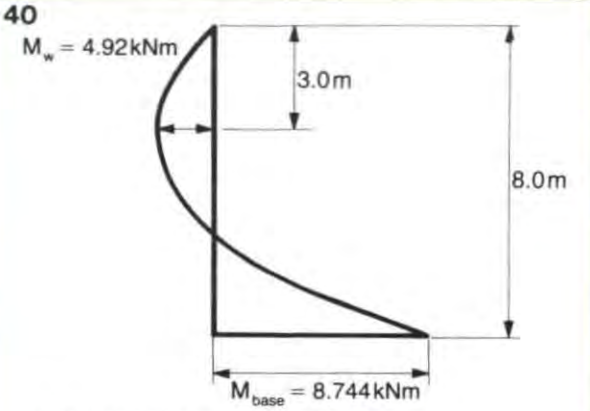
Approximate $MR_s = \gamma_f \times$ density $\times$ area $\times$ height $\times$ approx. lever arm

$= 0.9 \times 20 \times 0.230 \times 8 \times (0.475 \times 0.5575)$

$= 8.771$ kNm

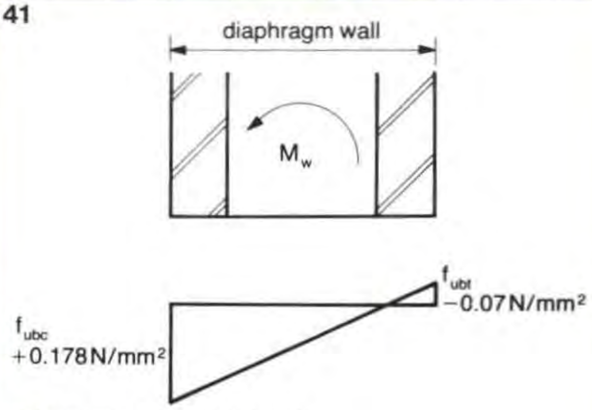
which is still greater than the design bending moment at base level and is extremely close to the value of MR_s calculated earlier as 9.099 kNm.

40



design bending moment diagram

41



design stresses at M_w level

7. Consider the stresses at level of M_w

The stress analysis at the level of M_w (maximum wall moment) assumes triangular stress distribution, using elastic analysis, but relates to ultimate strength requirements at the extreme edges of the wall face. This is considered to be a reasonable assumption when considering flexural tensile failure as against a rectangular stress block for flexural compression.

As with the calculation of stability moment of resistance, the design axial load comprises only the design own weight of the masonry therefore, at the level of M_w:

design axial load $= \gamma_f \times$ area $\times$ density $\times \frac{3}{8} \times$ height

$$= 0.9 \times 0.230 \times 20 \times \frac{3}{8} \times 8$$

$$= 12.42 \text{ kN/m}$$

then, from $\frac{\text{load}}{\text{area}} \pm \frac{\text{moment}}{\text{section modulus}}$

(i) design flexural compressive stress

$$f_{ubc} = \frac{12.42 \times 10^3}{0.230 \times 10^6} + \frac{4.92 \times 10^6}{39.69 \times 10^6}$$

$$f_{ubc} = 0.054 + 0.124 = +0.178 \text{ N/mm}^2$$

(ii) design flexural tensile stress

$f_{ubt} = 0.054 - 0.124 = -0.07$ N/mm² (see figure 41)

8. Allowable flexural stresses at M_w level

(a) allowable flexural tensile stress

$$p_{ubt} = \frac{f_{kx}}{\gamma_m}$$

where,

f_{kx} = 0.4 N/mm² for clay bricks with a water absorption of between 7 and 12% set in 1:1:6 mortar, taken from Table 3 of BS 5628: Part 1, for the plane of failure parallel to the bed joints.

γ_m =2.5 from Table 4 of BS 5628: Part 1 for special construction control and special manufacturing control of structural units.

Therefore,

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

which is greater than the calculated $f_{ubt} = -0.07\ N/mm^2$, hence the flexural tensile stresses are acceptable.

(b) allowable flexural compressive stress

$p_{ubc} = \frac{1.1\beta f_k}{\gamma_m}$

where,

f_k =9.41 N/mm^2 for bricks with a compressive strength of 41.5 N/mm^2 set in a designation (iii) 1:1:6 mortar interpolated from table 2a of BS 5628: Part 1.

γ_m =2.5 as above.

Calculate ß (for local buckling condition)

The effective length of the flange may be taken as 0.75 times the length of the internal void and the effective thickness as the actual thickness of the flange.

Hence, slenderness ratio (SR)$= \frac{0.75 \times 1.360}{0.1025} = 9.95$.

The centroid of the stressed area (being trapezoidal in shape) is unlikely to fall outside 0.05 of the thickness of the flange as an eccentricity although, at this stage, it cannot be accurately computed as the stress value has not yet been determined. To simplify the calculation, an eccentricity of 0.1 t_f will be catered for and with SR=9.95 and $e_x = 0.1\ t_f$, β is calculated from BS 5628: Part 1, table 7=0.88.

Therefore:

$p_{ubc} = \frac{1.1\beta f_k}{\gamma_m} = \frac{1.1 \times 0.88 \times 9.41}{2.5} = 3.644\ N/mm^2$

which is greater than the calculated $f_{ubc} = 0.178\ N/mm^2$, thus, the flexural compressive stresses are also acceptable.

9. Shear stress in cross-ribs

Reaction at base=design shear force V

$$V = \frac{5}{8} \times \gamma_f \times W_k \times h$$
$$= \frac{5}{8} \times 1.093 \times 8$$
$$= 5.47\ kN/m$$

Design shear stress, $v_h = K_1 V$ (see page 32 for K_1)

$$= \frac{20.52 \times 5.47}{10^3}$$
$$= 0.112\ N/mm^2$$

Therefore,

flexural shear force per brick course$=0.112 \times 75 \times 102.5 \times 10^{-3} = 0.86$ kN/m

10. Shear resistance

The shear resistance of 3 mm × 20 mm strip fishtail wall ties is given in table 8, BS 5628: Part 1 as $\frac{3.5}{3} = 1.167$ kN per tie.

Hence, ties are required in every course at the base of the wall, opening out as the height of the wall rises.

For a wall with bonded cross-ribs, alternate courses are taken through as headers on the flange elevation as shown in section in figure 42.

If the flexural shear force is considered to be resisted by the bonded headers only, the maximum shear force per brick at the base of the wall

$=0.112 \times 75 \times 2$ courses $\times 102.5 \times 10^{-3}$

$=1.722$ kN

BS 5628: Part 1 does not give shear strengths of bricks subject to this mode of shear loading. The

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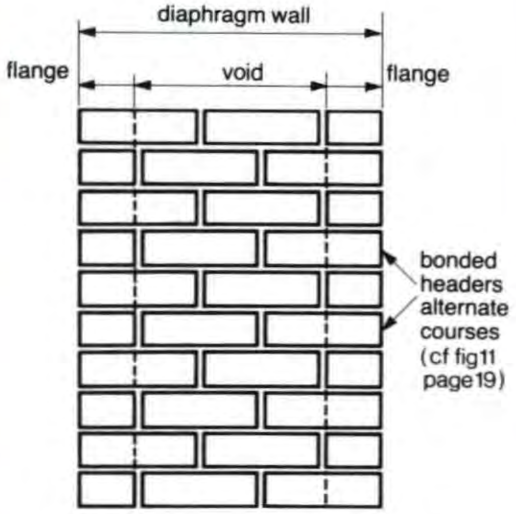

diaphragm walls showing shear resistance provided by bonded headers

characteristic shear strength given in clause 25 is for shear applied horizontally across a bed joint:

allowable shear stress $= \frac{f_v}{\gamma_{mv}} = \frac{0.35}{2.5} = 0.14\ \text{N/mm}^2$

which is a minimum value for designation (iii) mortar and is less than the design shear stress of 0.112 N/mm² calculated earlier. The alternate course bonded headers must provide greater shear resistance than the adjacent mortar joints. Research work is currently being undertaken by the authors and Dundee University with the support of the BDA to investigate the shear resistance of such a form of construction.

11. Roof plate and shear walls

The design of the roof plate and transverse shear walls is outside the scope of this design guide.

12. Check dead plus imposed plus wind loading combination

As stated in the calculation of loadings at the beginning of this design example, the most critical design condition is generally that of dead plus wind loading where the flexural tensile stresses are likely to be the limiting factor as has been demonstrated. The calculations will now proceed to check the dead plus imposed plus wind loading combination for which the flexural compressive stresses will be greater than for the dead plus wind combination, although it is anticipated that they will still remain comfortably within the allowable values calculated. The design will assume that the roof bearing is detailed in such a manner as to apply the roof loading on the centre-line of the wall section.

Design loads

dead + imposed + wind $= 1.2\ G_k + 1.2\ Q_k + 1.2\ W_k$
roof dead load $= 1.2 \times 0.67 = 0.804\ \text{kN/m}^2$
roof imposed load $= 1.2 \times 0.75 = 0.90\ \text{kN/m}^2$
wind loading walls $= 1.2 \times 0.781 = 0.937\ \text{kN/m}^2$
wind uplift on roof $= 1.2 \times 0.426 = 0.511\ \text{kN/m}^2$
Hence, dead plus imposed is greater than wind uplift.

Base wind moment

$$M_{base} = \frac{\gamma_f W_k h^2}{8} = \frac{0.937 \times 8^2}{8} = 7.496\ \text{kNm}$$

which is less than that for the dead plus wind loading combination previously calculated whereas, the stability moment of resistance, with the increased vertical loading, will be greater than previously calculated. Thus, the wall will be considered to act as a true propped cantilever for this loading combination also.

Wall wind moment

$$M_w = \frac{9\gamma_f W_k h^2}{128} = \frac{9 \times 0.937 \times 8^2}{128} = 4.216\ \text{kNm}$$

The design bending moment diagram for this loading combination is shown in figure 43.

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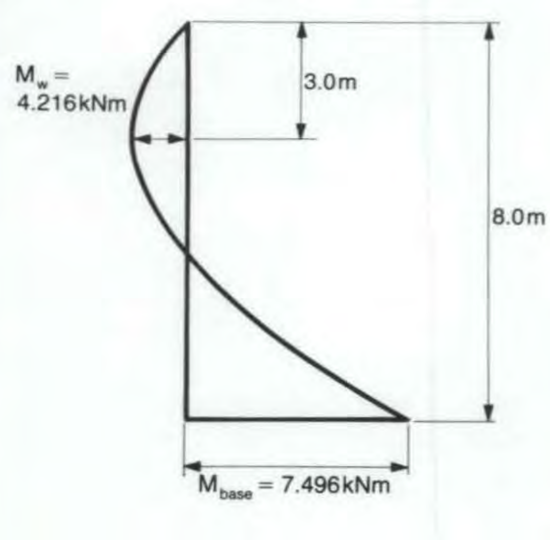

Stresses at M_w level

design dead + imposed at $\frac{3}{8}$ h (30 m span of roof beams)

roof dead load $= 0.804 \times \frac{30}{2} \qquad = 12.06$

imposed load $= 0.90 \times \frac{30}{2} \qquad = 13.50$

o.w. masonry $= 1.2 \times 0.23 \times 20 \times \frac{3}{8} \times 8 = 16.56$

total $= 42.12$ kN/m

Then, flexural compressive stress:

$$f_{ubc} = \frac{42.12 \times 10^3}{0.230 \times 10^6} + \frac{4.216 \times 10^6}{39.69 \times 10^6}$$

$$= 0.183 + 0.106$$
$$= 0.289 \text{ N/mm}^2$$

and flexural tensile stress:

$$f_{ubt} = 0.183 - 0.106$$
$$= +0.077 \text{ N/mm}^2$$

ie, there is compressive stress over the full width of the section as shown in figure 44.

These stress values are both within the previously calculated allowable values.

13. Check dead plus imposed loading combination
Finally, a check will be made on the overall stability of the wall and the associated maximum axial compressive stresses.

Design loads

dead plus imposed	$= 1.4\ G_k + 1.6\ Q_k$	
roof dead load	$= 1.4 \times 0.67$	$= 0.938$ kN/m^2
roof imposed load	$= 1.6 \times 0.75$	$= 1.200$ kN/m^2

o.w. masonry per m height $= 1.4 \times 0.23 \times 20 = 6.44$ kN/m height.

At base of wall, total design axial load:

roof dead load $= 0.938 \times \frac{30}{2} \qquad = 14.07$

roof imposed load $= 1.20 \times \frac{30}{2} \qquad = 18.00$

o.w. masonry $= 6.44 \times 8 \qquad = 51.52$

total $= 83.59$ kN/m

At mid-height of wall, total design axial load:

roof dead + imposed load	$=14.07+18.0$	$=32.07$
o.w. masonry	$=6.44\times4$	$=25.76$
	total	$=57.83$ kN/m

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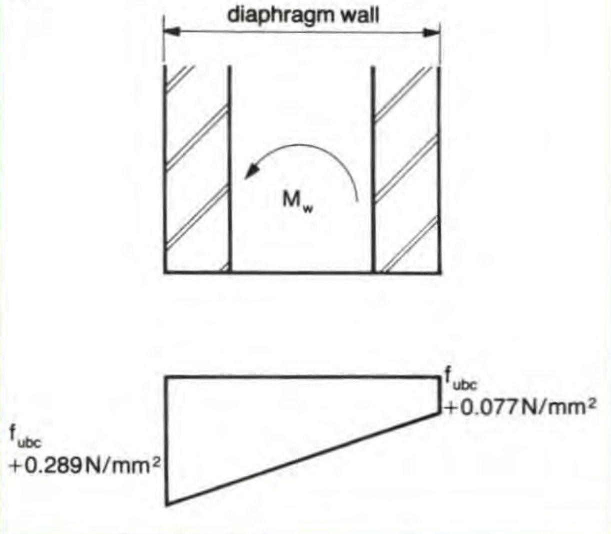

Calculate capacity reduction factor, β

eccentricity of loading, e_x $=0$ (as stated earlier)

slenderness ratio, SR $=\dfrac{\text{effective height}}{\text{effective thickness}}$

$=\dfrac{0.75\times8}{0.5575}$

$=10.6$

Hence, from BS 5628: Part 1, table 7, for $e_x=0$ and SR $=10.6$:

by interpolation, β $=0.955$

therefore, design vertical load resistance

$=\dfrac{\beta\times\text{area}\times f_k}{\gamma_m}$

$=\dfrac{0.955\times0.230\times9.41\times10^3}{2.5}$

$=826.76$ kN/m at mid-height of wall.

Which is far in excess of the total design axial load calculated as 57.83 kN/m, thus demonstrating that, in practice, usually only the dead plus wind load combination is designed.

At base of wall, maximum axial compressive stress

$=\dfrac{83.59\times10^3}{0.230\times10^6}$

$=0.363$ N/mm² at base of wall

allowable maximum axial compressive stress (ie, no slenderness reduction)

$=\dfrac{9.41}{2.5}$

$=3.764$ N/mm² at base of wall.

EXAMPLE no.2

Height of wall $=9.50$ m
wind pressure $=0.80$ kN/m²
from fig. 32, $K_2=1.30$ kN/m
from fig. 33, $Z=32.0\times10^3$ m³
Select wall section 7 (665 mm thick) and carry out a thorough analysis as shown in worked example no.1.

EXAMPLE no.3

Height of wall $=11.00$ m
wind pressure $=0.80$ kN/m²
from fig. 32, $K_2=1.67$ kN/m
from fig. 33, $Z=41.0\times10^3$ m³
Select wall section 10 (782.5 mm thick) and carry out a thorough analysis as shown in worked example no.1.

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