Design of concrete masonry diaphragm walls

Report of a Concrete Society Working Party
Foreword

Over the past ten to twenty years there has been a revival in the use of loadbearing masonry. This has been particularly noteworthy in the case of tall, single-storey structures, such as factories, warehouses, supermarkets and sports halls, which account for a large number of the buildings constructed in this country and abroad.

Prior to this revival the traditional masonry design solution for dealing with large vertical and horizontal loads was to increase the thickness of the masonry. This was wasteful of material, uneconomic in construction and ultimately led to the demise of the material as a structural medium and its replacement by concrete and steel. However, the introduction and development of the masonry diaphragm wall with its greatly improved slenderness ratio and section modulus has awakened the interest of a new generation of designers in the use of the system and a significant number of single-storey structures have been designed and built. Research, which has been underway for a number of years, is yielding valuable data for use where a conservative design approach has hitherto been necessary.

This guide provides the information which structural engineers need to design masonry diaphragm walls and includes two fully worked examples. It supplements BS 5628 Code of Practice for the use of masonry: Part 1 Structural use of unreinforced masonry which does not yet cover all aspects of the problem. The guide is based on the experience of the authors and will be of considerable help to designers seeking advantageous applications of a proven technique.
Summary
The Report is a design guide which provides the information required by structural engineers to design concrete masonry diaphragm walls. The sizes and types of concrete units which may be used for these walls are discussed and information is provided on aspects such as appearance, moisture penetration, movement joints, openings, services and acoustic and thermal properties. Key points on construction and economy are noted. Design principles are comprehensively explained and the background to the design method is fully detailed. Worked examples are provided covering a single-storey warehouse and a retaining wall. Design graphs and tables are provided and the report is illustrated by numerous sketches and figures.

Contents
1 Introduction, general arrangements and details ........................................ page 2
   1.1 Introduction
   1.2 General arrangements and details
      1.2.1 Wall section
      1.2.2 Wall/roof interaction
      1.2.3 Appearance and bonding
      1.2.4 Acoustic and thermal properties
      1.2.5 Moisture penetration
      1.2.6 Movement joints
      1.2.7 Openings in walls
      1.2.8 Services
      1.2.9 Foundations
   1.3 Construction
      1.3.1 Damp-proof courses and membranes
      1.3.2 Temporary propping
      1.3.3 Void cleaning
   1.4 Economics
   1.5 Other applications
   1.6 Potential for development and requirements for research

2 Design principles ........................................ 8
   2.1 Design symbols
   2.2 Lateral loading
      2.2.1 Determination of centres of cross-ribs
      2.2.2 Typical design dimensions
   2.3 Properties of sections
      2.3.1 Concrete blockwork
      2.3.2 Concrete brickwork
   2.4 Assumed behaviour of a diaphragm wall
      2.4.1 Stability moment of resistance (MRs)
      2.4.2 Design bending moments
      2.4.3 Allowable flexural stresses
      2.4.4 Trial section coefficients $K_a$ and $Z$
      2.4.5 Shear stress coefficient $K_1$
   2.5 Vertical loading only
      2.5.1 Slenderness ratio
      2.5.2 Effective height
      2.5.3 Effective thickness
      2.5.4 Eccentricity of vertical loading
      2.5.5 Capacity reduction factor $\beta$

3 Design procedure and worked examples .......... 21
   Worked example 1: Warehouse building
   Worked example 2: Retaining wall in unreinforced masonry (uncracked section)

4 Bibliography ............................................. 29
1 Introduction, general arrangements and details

1.1 Introduction
Developments in structural masonry, particularly the introduction of diaphragm and fin walls over the past 20 years, have provided economically viable alternatives to reinforced concrete, structural steel and structural timber for certain applications. Diaphragm walls are now acknowledged as having many advantages to offer the client together with benefits for designers, developers, contractors and users, compared with framed solutions. Nor is their use limited to tall single-storey buildings; diaphragm walls have been successfully applied to other types of construction such as retaining walls and boundary/barrier walls. Indeed, wherever large lateral loadings or moments are combined with minimal vertical loading, the diaphragm wall is worthy of consideration.

Diaphragm walls fulfil the functions of structure, cladding, insulation and lining in one operation, using only one trade, which can be carried out by the main contractor. Experience has shown that diaphragm walls are well suited to the building types mentioned in the foreword, having proved to be more economic, speedier and simpler to construct, and more durable than the traditional steel frame and sheet cladding type of building. There is scope to exploit the aesthetic qualities of concrete masonry and its robustness offers vandal resistance as a further bonus.

Generally, diaphragm walls become more economic compared to framed construction, as the height of the wall increases. On recent projects relatively narrow diaphragm walls with a 100 mm wide cavity have also proved to be economic both in construction time and financial terms for buildings with wall heights of only about 4.5 m. This wall height is ideally suited to small factory units and numerous such projects, some incorporating crane gantries, have been built throughout the country. To date, diaphragm walls with heights of up to 10 m have been designed and there is no reason to suppose that this is either the structural or the economic limit. Diaphragm walls have no advantage however where normal cavity walls can meet all the structural requirements.

Diaphragm walls used in tall single-storey buildings illustrate many of the advantages of the technique. They also demonstrate the overall design approach which can produce structural economies by considering the interaction of different parts of the structure, for example walls and roof, as opposed to considering these elements in isolation. This is extremely important in the design of structural masonry. The design and construction of tall single-storey buildings using diaphragm walling is therefore considered in some detail to illustrate these principles.

The correct and economical engineering use of any material requires a full understanding of its properties. Concrete masonry is no exception. It is strong in compression and comparatively weak in tension and it is therefore ideal for supporting axial compressive loads, but cannot so readily resist lateral loads which induce tensile bending stresses. The development of diaphragm walls began with the realization of the importance of the use of a high ratio of section modulus to area (Z/A) and the need to take advantage of the gravitational forces involved. Both of these requirements involve an appropriate geometric distribution of the material, that is to distribute the material to its largest practical lever arm. It is also necessary of course to provide adequate resistance to shear forces and to the buckling tendency of the compression portion of the section.

By applying the basic principles of high Z/A ratio and large lever arm and by using a minimum thickness for leaves and cross-ribs, as appropriate to the loading condition, diaphragm wall construction has evolved and developed into an economic structural medium.

From the practical point of view, the geometric arrangements of the various wall profiles should be consistent with standard block or brick dimensions. In this regard concrete masonry diaphragm wall sections offer an almost unlimited combination of profiles resulting from the wide range of sizes and thicknesses of commonly available standard units. Hence, with a careful combination of bonding requirements and structural design requirements, the designer should be able to achieve maximum efficiency and economy from a diaphragm wall design in concrete masonry.

1.2 General arrangements and details
1.2.1 Wall section
A diaphragm wall is basically a wide cavity wall with the leaves of concrete masonry connected together with perpendicular cross-ribs (diaphragms) to form a series of ‘box’ or I sections as shown in Figure 1.

The two 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. The distance between flanges is designed to suit the individual requirements of each project, taking account of unit size and bonding, and should be kept to a minimum to save on space and cost. The spacing of cross-ribs is determined from a number of factors including the need for the flange to span horizontally between them. The spacing and thickness of the cross-ribs are also subject to shear resistance requirements.

1.2.2 Wall/roof interaction
The concrete masonry is in this context no longer mere cladding to steel columns but part of the structure. It must resist both lateral load from the wind and vertical load from the roof. The vertical load is unlikely to be significant since both the self weight (of lightweight roofs) and the superimposed loads are generally low. The critical load in the case of tall single-storey buildings is usually the lateral load due to wind. The wall can act as a vertical free cantilever or as a propped cantilever as shown in Figure 2. The bending moment at the base of the propped cantilever is shown to be only a quarter of that for a free cantilever. In order to obtain the maximum economy in the overall cost of the structure, the roof of a diaphragm wall structure should be used, wherever possible, to prop or tie the tops of the walls. The resulting horizontal reactions can be transferred through the roof structure to the gable or other transverse walls which then act as shear walls in resisting these forces.

Often the decking material itself, if suitably stiff and fixed, can be used as a plate in conjunction with the main roof beams. But where this is not the case, a horizontal wind girder may be formed utilizing the concrete capping beam as described below. The insertion of light diagonal bracing between the capping beam and perhaps a purlin acting as the other boom is often all that is necessary.

The roof deck can be of a variety of materials and supported in many ways. Depending on the spans involved, the most economic roof may be steel beams or girders spaced at centres to suit the most economical arrangement. On very long spans, a steel space deck may prove to be economic in providing the necessary support and can be designed to act as a plate to transfer the propping loads to the transverse walls. Timber decking on glued laminated timber beams has also been used in which the decking has proved adequate in providing the horizontal plate to prop the top of the wall.

Precast, prestressed concrete purlins supporting woodwool decking may be economic but this type of roof is likely to require a secondary bracing system to provide the horizontal plate action.

A reinforced concrete capping beam can be used at the
top of the diaphragm wall to transfer the propping forces into the roof deck. It can also serve as a counterweight to overcome uplift forces from wind suction acting on lightweight decking and the same capping beam may also be used as the boom member of the roof plate.

The capping beam may be constructed either in situ concrete or by precasting the beam in bay lengths, using suitable connections to transfer the forces at the joints. The design of these beams should provide for the horizontal forces which must be transferred from the wall to the roof structure, but this is outside the scope of this guide. The precast beams may be cast and supplied by the block manufacturer and can be used as the seating and fixing for the roof structure, as shown in Figure 3. Generally the more successful and popular method of construction is that of precasting. This overcomes the problem of keeping the facing masonry clean, avoids possible formwork difficulties and reduces the on-site construction time. If in situ beams are used the shuttering can be retrieved by leaving one of the wall leaves down a couple of courses and building up later. Alternatively, permanent formwork can be used.

1.2.3 Appearance and bonding
The wall/roof junction of the building can be treated in a variety of ways, for example as illustrated in Figure 3. It is also unnecessary for the diaphragm wall to be designed with flat faces on each side, for example a fluted arrangement can be neatly incorporated in the structure as shown in Figure 4. The treatment of the face of the wall and its junction with the roof gives scope for a wide variety of interesting external and internal elevations.

It is possible to use either bonded joints, between the cross-ribs and leaves, or butt joints, with designed shear ties. The choice is often governed by cost and appearance.

In concrete masonry diaphragm walls there are a number of points to consider which will generally result in the cross-ribs being tied rather than bonded, e.g.:

(a) effect of bonding on the rate of construction;
(b) different sized units such as a concrete brick external leaf and a concrete block internal leaf;
(c) rain and damp penetration which, if the external face is not to be rendered, may require a vertical damp-proof course at the junction of the cross-rib and the leaf (see Section 1.3.1);
(d) differing materials used for internal and external leaves resulting in varying degrees of shrinkage movement or even opposing movements (see below).

An architect may wish to design the building using clay bricks externally and concrete blocks internally. This method of construction should generally be avoided as concrete products tend to shrink and clay products tend to expand with time, causing increased stresses at the interface of the two materials. If combinations of such dissimilar materials cannot be avoided, shear-tied joints should always be used.

1.2.4 Acoustic and thermal properties
Statutory requirements for sound insulation rarely apply to diaphragm wall buildings. There can be occasions where it is desirable to use the walls as a sound barrier and whilst no actual tests have been carried out, experience has shown that diaphragm walls perform well.

Low strength concrete masonry can provide an inherently high level of thermal insulation. However, where this is not adequate, it is a simple matter to improve its performance by fixing insulation boards or quilts to the void face of the inner leaf as shown in Figure 5.

Alternatively, the designer may choose to omit the insulation from within the wall and to ‘trade off’ against an improved insulation within the roof space. This is a commonly employed design technique within the scope and requirements of the current Building Regulations, Part FF3. Research carried out by the Building Research Establishment confirmed that the dew point will normally occur within the cross-ribs or voids and that condensation or pattern-staining on the inner leaf is very unlikely to occur. This view concurs with observations of
the actual performance of various buildings over the last 15 years.

1.2.5 Moisture penetration
The possibility of moisture being transmitted to the inside face of the wall depends on a number of factors including the workmanship and the physical characteristics of the concrete masonry units. One overriding factor appears to be the width of the wall between flanges. Moisture which penetrates into the cross-rib can dry out before it reaches the inner leaf provided there is adequate ventilation within the voids and the width of the wall is sufficient to allow this process to take place. Furthermore, the function of some buildings is such that occasional penetration can be tolerated.

In many cases when the diaphragm wall forms the external envelope of a building, the overall thickness of the wall will be sufficient to prevent moisture penetration. However, in some severe exposure areas and/or when the possibility of damp penetration to the inner face is particularly critical it is recommended that a vertical damp-proof membrane be provided at the junction of the cross-rib with the external flange. Alternatively, the external face may be rendered. In very severe exposure areas both these measures may be required.

Retaining walls are a special case where moisture penetration is either not critical or is overcome, for example, by providing a damp-proof membrane covering the inner or outer wall surface.

1.2.6 Movement joints
Movement joints are required at the appropriate centres, in accordance with the normal recommendations for concrete masonry given in CP 121 and manufacturers’ technical publications. Joints to accommodate shrinkage movement of the concrete masonry are usually necessary and may be achieved by providing double cross-ribs, one either side of each joint, as in Figure 6.

Such joints should also be provided near to a corner to allow the wall at this location to hinge at the base. It is not usually necessary for the joints in the masonry to extend through the concrete capping beam, if provided.

1.2.7 Openings in walls
Large openings for access, windows, service entries, etc., can create high local loading conditions from both horizontal wind loading and increased axial loads at lintel bearings. The opening can be accommodated by providing a beam or lintel to carry the vertical load and by using extra or thicker cross-ribs on each side of the openings to carry the increased lateral load, as in Figure 7. Double cross-ribs are preferred at openings through which vehicles are likely to pass; the outer cross-rib is often considered to be non-structural, using the inner cross-rib for the lintel support. Vertical damp-proof membranes should be provided at external openings in diaphragm walls. The use of lintel blocks or bond beam blocks, commonly available from numerous concrete block manufacturers, to span openings in diaphragm walls, may be considered. Bed-joint reinforcement may be included around wall openings to minimize the effects of shrinkage and thermal movement.
1.2.8 Services
The large vertical voids within diaphragm walls are useful to accommodate certain services. The size and location of access openings may cause high local stressing in the concrete masonry and these must be considered in the design. Service ducts can be positioned within the voids as shown in Figure 8. Such ducts should, of course, be ventilated when housing gas pipes.

1.2.9 Foundations
The load from a wall in a tall single-storey building is transferred to the foundations as a uniformly distributed load combined with the applied bending moment and not as a point load as is the case with framed construction. Thus the ground contact pressures are sufficiently low for a normal concrete strip footing to be adequate (see Figure 9). This must, of course, be checked by considering the relevant soil properties. The designer should include in the foundation design a check on the effect of the applied moment at the base of the diaphragm wall. The calculation of the magnitude of this moment is discussed later in this design guide and in the worked examples.

1.3 Construction
1.3.1 Damp-proof courses and membranes
Horizontal damp-proof courses should be chosen which give the necessary shear resistance to prevent sliding and which do not squeeze out under load. In situations where vertical damp-proof membranes between leaf and cross-ribs are considered necessary, an unbonded diaphragm wall construction should be used. It is essential that the vertical damp-proof membrane does not prevent or inhibit the tying of the cross-ribs to the leaves. In practice, brush-applied types of damp-proof membrane and metal shear ties have proved to be a successful combination. Fully-filled, flush mortar joints are necessary for the effective application of a brush-applied damp-proof membrane treatment. The durability of the shear ties should be consistent with the design life of the structure.

Vertical damp-proof membranes should, as stated earlier, be provided at door and window openings and it is recommended that a double cross-rib be provided to accommodate this, as in Figure 7.

1.3.2 Temporary propping
The diaphragm wall, like most other walls, is often in its most critical state during construction, before the roof has been constructed and fixed, particularly if the design is based on roof propping action. The contractor must take the usual precautions for temporary restraint such as the use of scaffolding to prop the walls to ensure that they remain stable, or by other means. Better workmanship is achieved by working from scaffolding on both sides of the diaphragm wall rather than working overhand from one side. The double scaffold system is usually all that is necessary to provide adequate temporary propping to the walls, but this should be checked by structural calculation.

1.3.3 Void cleaning
It is equally important in diaphragm wall construction, as in conventional cavity work, to maintain high standards of workmanship to ensure satisfactory performance of the wall. The voids should be clean, but elaborate cleaning out methods are not usually necessary with diaphragm wall construction owing to the larger voids employed.

1.4 Economics
Economy is generally the most cogent reason for adopting any particular building method. In the authors' experience this has always been the prime factor for adopting the diaphragm wall method of construction. Considerable evidence has shown that, for the more sophisticated type of structure, such as sports halls, theatres and swimming pools, the diaphragm wall is the most economic.

Recently, for basic industrial structures, this building method has provided the most cost-effective solutions. Experience has shown that diaphragm wall projects can produce a considerable saving of time, both in the pre-contract period and during the on-site construction. This is largely because the design, estimating and
tendering procedures are simplified, and dependence on a variety of materials, subcontractors and separate site operations is eliminated, thus enabling the general contractor to work to a tighter programme under his own close control.

1.5 Other applications
At first, diaphragm walls were mainly used for tall single-storey buildings, in particular sports halls in schools and leisure centres. Their success has led to architects, engineers, developers and contractors applying the technique to numerous other types of buildings. Among these are now included factories, warehouses (with and without overhead travelling cranes), garages, churches, theatres, assembly halls, squash courts and retaining walls.

There are many applications in other fields, particularly where lateral loading is more significant than vertical loading. A diaphragm wall has been used as a mass retaining wall on a site which was covered with a large amount of demolition rubble. The rubble was used to fill the voids, and a cheap, strong, mass-retaining wall was achieved as shown in Figure 10.

A variation on this basic application for retaining walls is shown in Figure 11. The design can use a plain diaphragm wall, or further benefits may be derived by post-tensioning or reinforcing a similar wall section. Sound reflectors on motorways in urban areas are another application for diaphragm walls. At present some reflectors are constructed in precast concrete, steel or timber. It is believed that concrete masonry diaphragm walls would be cheaper and more durable, and provide greater aesthetic appeal and potential for such applications. They may also be used for fire barriers in industrial buildings, and in farm buildings as silos or bins for the storage of grain, potatoes, etc.

Post-tensioned diaphragm walls were used in the Oak Tree Lane Community Centre, Mansfield, Nottinghamshire, which was subjected to massive ground subsidence due to coal extraction. The stiff composite structure performed excellently where other forms of construction might have suffered considerable damage due to the subsidence. This particular technique was recognized by the receipt of a Certificate of Merit for the project in the 1982 Structural Design Awards Scheme organized by the Brick Development Association.
A similar post-tensioning technique was used in a concrete blockwork diaphragm wall project where the architect required clerestory windows around the full perimeter of the building. This meant that propping the wall by means of the roof structure was not possible and thus the wall was designed as a free cantilever. Post-tensioning the diaphragm helped it to cope with the higher stresses from the increased bending moment. Whilst most of the applications of diaphragm walls have tended to concentrate on their effectiveness in resisting lateral loading, they also possess ideal properties to carry heavy axial loads. This results from their robustness for resisting buckling, thus allowing them to be designed at efficient stress levels to support a heavy axial load.

1.6 Potential for development and requirements for research
The development potential of the basic diaphragm wall technique is only now beginning to be realized. The diaphragm section with its high Z/A ratio and high radius of gyration make it ideal for prestressing. By prestressing masonry sections, further advantage can be made of masonry’s high compressive strength whilst overcoming the disadvantage of its low tensile strength. Several prestressed masonry diaphragm walls have now been constructed, some including the use of concrete blockwork.

Concrete blockwork, in particular, lends itself to reinforcing and there is considerable potential for the development of this technique. Reinforcement may be applied to the wall as a whole or to strengthen one leaf of the wall, for instance in a retaining wall, to enable a comparatively thin leaf to span between the cross-ribs to support high lateral pressures.

The ease of construction and comparative lack of sophistication make concrete masonry diaphragm walls particularly effective in producing a highly engineered structure with a work force possessing normal trade skills. This obviously has considerable potential when considering agricultural buildings, construction in remote areas, and developing countries.

There are a number of aspects where engineering research is required to enable the diaphragm wall technique to be more fully exploited and extended in its use and range of applications. One such aspect concerns the shear at the junction of the cross-rib and flange. In bonded forms of construction, data are required for concrete bricks and more particularly concrete blocks, so that where very high shear stresses occur, for example in retaining walls, more accurate assessments of the shear strength may be made. In tied construction, where hollow or cellular blocks are used, information is required on the shear resistance which may be developed with the metal ties built into the external shell of those hollow units.

2 Design principles
This design guide is written in limit state terms and follows BS 5628 where appropriate. However, the design of more complex geometric masonry elements, such as the diaphragm wall, is not fully covered in BS 5628 and supplementary design procedures have been introduced. These are discussed and developed in the text before applying them later to the worked examples.

The design of the earliest diaphragm walls was made using fundamental engineering principles. The research work which has been completed to date has confirmed the logic of early assumptions and engineers can approach future designs of diaphragm walls with confidence.

It is only rarely that compressive stresses in the masonry are the governing factor in the design of diaphragm walls in tall single-storey buildings. The limiting condition is bending tensile stress due to wind loading, and this consideration determines the spacing of the leaves (or flanges). As a result, concrete masonry of low compressive strength is usually adequate for loadbearing purposes and the final choice of masonry type is more generally related to aesthetics, durability and to thermal, sound and fire resistance properties.

The structural calculations are carried out by selecting a trial section which is then checked for the various stress conditions. Full discussion and worked examples are included in Section 3 of this design guide.

2.1 Design symbols
The design procedure introduces additional symbols to those provided in BS 5628: Part 1. A full list of all the symbols used in the text and the worked examples is given below. The additional symbols have been marked with an asterisk * for identification.

\[ A \] horizontal cross-sectional area

\[ B \] distance between centres of cross-ribs

\[ b_1 \] thickness of cross-rib

\[ b_2 \] length of void between cross-ribs

\[ C_{e} \] external pressure coefficient (wind)

\[ C_{p} \] internal pressure coefficient (wind)

\[ D \] overall thickness of diaphragm wall

\[ d \] width of void between flanges

\[ e \] eccentricity at top of wall

\[ f_c \] characteristic compressive strength of masonry

\[ f_{cx} \] characteristic flexural strength of masonry

\[ f_{t} \] characteristic shear strength of masonry

\[ f_{u} \] applied flexural compressive stress at design load

\[ f_{u} \] applied flexural tensile stress at design load
2.2 Lateral loading

2.2.1 Determination of centres of cross-ribs

The spacing of the cross-ribs is generally governed by one of the following four conditions:

(a) The flanges and cross-ribs must act together compositely as shown in Figure 12. The length of
flange must be restricted in accordance with Clause 36.4.3, BS 5628: Part 1.

(b) In developing the bending resistance of the composite box section, the interface between flange
and cross-rib must not fail in flexural shear, as shown in Figure 15.

Note: Condition (c) does not limit the maximum centres at which the cross-ribs may be spaced but does limit
the extent of the leaves which can be considered as contributing to the flange of the I section for structural
purposes.

Typical calculations for the cross-rib centres from these limiting conditions for a tall single-storey building give
the following:

Condition (a) (refer to Figure 12):

\[
M = P_{ub} \frac{Z}{2} \]

in which

\[
M = \text{applied bending moment due to wind} = \gamma_I W_k B_d^2 / 10 \quad \text{(reduced moment for continuity)}
\]

\[
P_{ub} = \text{allowable flexural tensile stress} = f_{yk} / \gamma_m
\]

\[
Z = \text{section modulus} = t_s^2 / 6 \quad \text{per unit height}
\]

Then, typically assuming

\[
W_k = 0.6 \text{ kN/m}^2
\]

\[
t_s = 0.1 \text{ m}
\]

\[
B_d = \text{cross-rib centres required}
\]

\[
f_{yk} = 0.6 \text{ N/mm}^2 \text{ i.e. } 7 \text{ N/mm}^2 \text{ block set in a designation (iii) mortar (see BS 5628: Part 1, Table 3)}
\]

\[
\gamma_m = 3.5 \quad \text{(BS 5628: Part 1, Table 4)} \text{ and } \gamma_I = 1.4
\]

\[
M = \gamma_I W_k B_d^2 / 10 = 1.4 \times 0.6 \times B_d^2 / 10 = 0.084 B_d^2 \text{ kNm}
\]

\[
P_{ub} = f_{yk} / \gamma_m = 0.6 / 3.5 = 0.17 \text{ N/mm}^2
\]

\[
Z = t_s^2 / 6 = 0.1^2 / 6 = 1.67 \times 10^{-3} \text{ m}^3
\]

But \[
M = P_{ub} Z
\]

\[
0.084 B_d^2 = 0.17 \times 10^3 \times 1.67 \times 10^{-3} \text{ (kN and m units)}
\]

\[
\therefore B_d = 1.84 \text{ m}
\]

For a retaining wall, subjected to greater lateral loading, this condition can be the most critical and it is often
necessary to increase the leaf and cross-rib thicknesses and/or reduce the cross-rib spacing from those normally
encountered in tall single-storey buildings.
Condition (b) (refer to Figure 13):

Slenderness ratio \( \frac{h}{t_l} = \frac{B_a}{t_l} \)

Maximum permitted slenderness ratio = 27 (BS 5628: Part 1, Clause 28.1)

\[ B_a/t_l = 27 \]

\[ \therefore B_a = 27 \times 0.1 = 2.7 \text{ m} \]

Condition (c) (refer to Figure 14):

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 six times the thickness of the flange where the flange is continuous, but in no case more than half the distance between the piers – in this design the cross-ribs may be taken to be the piers.

Then, typically, for a 6 m high diaphragm wall constructed throughout in 100 mm thick units,

\[ B_a = 6t_l + 6t_l + b \]

\[ t_l = b, \text{ (see Figure 16)} \]

\[ \therefore B_a = 1.30 \text{ m} \]

It is considered that the effective flange width should also be limited to a proportion of the height of the wall even though no such limitation is provided for in BS 5628: Part 1. It is therefore proposed that one-third of the wall height, as was applicable in CP 114: 1969, Clause 311(e), would be an acceptable limit and will be used here.

Then:

\[ B_a = \frac{h}{3} = \frac{6}{3} = 2.0 \text{ m} \]

Condition (d) (refer to Figure 15):

There are two methods of achieving shear resistance between the leaf and the cross-rib, i.e. by bonding the brick and block courses or by using designed metal shear ties to maintain the composite action of the box section (see Figure 17). The choice of method is determined by a number of considerations which are discussed in Section 1.2.3.

2.2.2 Typical design dimensions

Experience has shown that in the case of single-storey buildings with wind forces of around 0.6 kN/m² and wall heights in the order of 8 m, the cross-rib centres are generally 1.0 to 1.3 m. The cross-ribs may be spaced at wider centres provided that only a restricted length of flange is considered as the effective section in resisting the bending, and that all other stress criteria are satisfied.

The greater the overall thickness of the wall, the greater is its resistance to lateral wind forces. Increasing its overall thickness also improves the wall’s slenderness ratio, and thus its axial loadbearing capacity. Again, experience has shown that with the wind forces and wall heights quoted above, the overall thickness of the wall needs to be 0.4 to 0.7 m as shown in Figure 18, although the actual dimensions must be determined by calculation as shown later in the worked example.

2.3 Properties of sections

The range of overall thicknesses which can be achieved with concrete masonry diaphragm walls is considerable and affords the designer maximum flexibility.

In order to simplify the presentation of section properties a selected number of wall profiles have been calculated and their properties tabulated in Tables 1 and 2. Table 1 deals with concrete blockwork diaphragms in
which only solid blocks with the most commonly used work size, 440 x 215 x 100 mm, are considered. Table 2 deals with concrete brickwork diaphragms.

**Note:** Other standard sizes of concrete masonry unit including voided units, e.g. hollow or cellular units, may be used.

### 2.3.1 Concrete blockwork

Figure 19 shows three typical arrangements for achieving bonded cross-ribs from which the various bonding profiles and their respective section properties can be developed. For unbonded cross-ribs the centres are less restricted but for convenience they are often placed on the centre of a block or at a perpend joint.

Figure 20 shows the combinations of wall depths and cross-rib spacings upon which Table 1 is based. In this figure and in the table, three depths of tied walls and two depths of bonded wall are considered. The calculations of the section properties of wall reference A are set out below:

**Wall reference A (440 x 215 x 100 mm work size of blocks):**

- \( D = 215 + (2 \times 10) + (2 \times 100) = 435 \text{ mm} \)
- \( d = 435 - (2 \times 100) = 235 \text{ mm} \)
- \( B_d = (2 \times 440) + (2 \times 10) = 900 \text{ mm} \)

<table>
<thead>
<tr>
<th>Section</th>
<th>Dimensions (m)</th>
<th>( D )</th>
<th>( d )</th>
<th>( B_d )</th>
<th>( b_v )</th>
<th>Section properties per diaphragm</th>
<th>Section properties per metre</th>
<th>Shear stress coefficient ( K_1 ) (kN/m)</th>
<th>Stability moment ( K_2 ) (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.435</td>
<td>0.235</td>
<td>0.90</td>
<td>0.80</td>
<td>5.31</td>
<td>24.40</td>
<td>0.2035</td>
<td>28.39</td>
<td>0.63</td>
</tr>
<tr>
<td>B</td>
<td>0.435</td>
<td>0.235</td>
<td>1.125</td>
<td>1.025</td>
<td>6.60</td>
<td>30.30</td>
<td>0.2485</td>
<td>28.55</td>
<td>0.616</td>
</tr>
<tr>
<td>C</td>
<td>0.435</td>
<td>0.235</td>
<td>1.350</td>
<td>1.250</td>
<td>7.90</td>
<td>36.00</td>
<td>0.2935</td>
<td>28.62</td>
<td>0.605</td>
</tr>
<tr>
<td>D</td>
<td>0.550</td>
<td>0.350</td>
<td>0.730</td>
<td>0.630</td>
<td>7.87</td>
<td>28.62</td>
<td>0.181</td>
<td>21.24</td>
<td>0.817</td>
</tr>
<tr>
<td>E</td>
<td>0.550</td>
<td>0.350</td>
<td>1.180</td>
<td>1.080</td>
<td>12.50</td>
<td>45.46</td>
<td>0.271</td>
<td>21.24</td>
<td>0.817</td>
</tr>
<tr>
<td>F</td>
<td>0.660</td>
<td>0.460</td>
<td>0.900</td>
<td>0.800</td>
<td>15.00</td>
<td>45.67</td>
<td>0.226</td>
<td>16.74</td>
<td>1.06</td>
</tr>
<tr>
<td>G</td>
<td>0.660</td>
<td>0.460</td>
<td>1.125</td>
<td>1.025</td>
<td>18.00</td>
<td>56.30</td>
<td>0.271</td>
<td>16.50</td>
<td>1.06</td>
</tr>
<tr>
<td>H</td>
<td>0.660</td>
<td>0.460</td>
<td>1.350</td>
<td>1.250</td>
<td>22.20</td>
<td>67.70</td>
<td>0.316</td>
<td>16.40</td>
<td>1.06</td>
</tr>
<tr>
<td>J</td>
<td>0.775</td>
<td>0.575</td>
<td>0.730</td>
<td>0.630</td>
<td>18.30</td>
<td>47.32</td>
<td>0.204</td>
<td>25.07</td>
<td>1.387</td>
</tr>
<tr>
<td>K</td>
<td>0.775</td>
<td>0.575</td>
<td>1.180</td>
<td>1.080</td>
<td>28.60</td>
<td>73.80</td>
<td>0.2935</td>
<td>24.20</td>
<td>1.387</td>
</tr>
<tr>
<td>L</td>
<td>0.885</td>
<td>0.685</td>
<td>0.900</td>
<td>0.800</td>
<td>30.50</td>
<td>68.90</td>
<td>0.2485</td>
<td>33.80</td>
<td>1.566</td>
</tr>
<tr>
<td>M</td>
<td>0.885</td>
<td>0.685</td>
<td>1.125</td>
<td>1.025</td>
<td>37.50</td>
<td>84.70</td>
<td>0.2935</td>
<td>33.30</td>
<td>1.480</td>
</tr>
<tr>
<td>N</td>
<td>0.885</td>
<td>0.685</td>
<td>1.350</td>
<td>1.250</td>
<td>44.40</td>
<td>99.40</td>
<td>0.3385</td>
<td>32.00</td>
<td>1.42</td>
</tr>
</tbody>
</table>
Figure 19: Typical arrangements when cross-ribs are bonded into flanges
Figure 20: Wall depths and cross-rib spacings used for Table 1 data
2.3.2 Concrete brickwork

Figure 21 shows some typical bonded concrete brick diaphragm wall profiles which have been found useful in practice. They are based on the standard brick format. The calculations for the structural properties of these sections are made in the manner described in Section 2.3.1. Table 2 presents the properties of a number of such walls of varying depth and cross-rib spacing.

2.4 Assumed behaviour of a diaphragm wall

The critical loading condition for single-storey buildings is generally that created by lateral loading from wind forces. The vertical loading condition is rarely significant by itself and on the basis of engineering judgement may often be eliminated from the design.

The limiting stresses for lateral loading are usually on the tensile face of the wall. The geometric profile of the diaphragm wall is well suited to offering a high resistance to these limiting stresses.

For most buildings, and particularly tall single-storey buildings, the roof is designed and detailed to act as a prop or tie to the head of the external walls. This permits the wall design to be similar to that of a propped cantilever, which generally results in the maximum structural efficiency and therefore economy.

In a propped cantilever design approach there are two locations of critical bending moment to consider (see Figure 22):

- level A – at the base of the wall (which is generally at dpc level) where it is recommended that the wall is designed as a cracked section.
- level B – at a level approximately 3/8 h down from the top of the wall where the wall is designed as an uncracked section.

The resistance at these two levels of critical bending moment is provided by:

at level A – the stability moment of resistance ($MR_s$) of the cracked wall;

at level B – the flexural tensile resistance of the wall.

By treating level A as a 'cracked section' the design is unaffected by any lateral movement of the prop which may occur due to lateral deflection of the roof plate.

2.4.1 Stability moment of resistance ($MR_t$)

Single-storey buildings tend to have a lightweight 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 net 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 23. The resistance moment is generated by the self-weight of the wall (together with any net roof loads) acting over a lever arm of approximately half the thickness of the wall from the centre of rotation.

In limit state design the centre of rotation is taken as the centroid of a rectangular stress block and not as a knife edge, i.e. the extreme edge of the wall. Thus the lever arm is calculated as the distance between this centroid and the resultant of the downward vertical loads.

The width of the rectangular stress block is calculated as the width of masonry as shown in Figure 24, stressed to ultimate. This produces the maximum lever arm to generate the maximum stability moment of resistance $MR_t$.

It is convenient to approximate this lever arm value for the purpose of obtaining an initial trial section and, for this purpose only, a lever arm value of 0.475D has been found from experience to be a reasonable approximation. This approximate lever arm is shown in Figure 25 and its application is discussed in Section 2.4.5.

---

### Table 2. Section properties – concrete bricks of work size 215 x 102.5 x 65 mm.

<table>
<thead>
<tr>
<th>Section</th>
<th>$D$ (m)</th>
<th>$d$ (m)</th>
<th>$B_s$ (m)</th>
<th>$b_t$ (m)</th>
<th>$I_{x10^{-3}}$ (m³)</th>
<th>$I_{x10^{-3}}$ (m³)</th>
<th>$A$ (m²)</th>
<th>$A$ (m²)</th>
<th>Shear stress coefficient $K_s$ (kN/m)</th>
<th>Stability moment $K_t$ (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.44</td>
<td>0.236</td>
<td>1.4625</td>
<td>1.36</td>
<td>8.91</td>
<td>8584</td>
<td>595.7</td>
<td>383.4</td>
<td>28.76</td>
<td>0.222</td>
</tr>
<tr>
<td>2</td>
<td>0.44</td>
<td>0.236</td>
<td>1.375</td>
<td>1.135</td>
<td>7.65</td>
<td>34.23</td>
<td>27.8</td>
<td>36.49</td>
<td>27.33</td>
<td>0.225</td>
</tr>
<tr>
<td>3</td>
<td>0.44</td>
<td>0.236</td>
<td>1.0125</td>
<td>0.91</td>
<td>6.21</td>
<td>28.63</td>
<td>23.22</td>
<td>37.8</td>
<td>27.88</td>
<td>0.229</td>
</tr>
<tr>
<td>4</td>
<td>0.5575</td>
<td>0.352</td>
<td>1.4625</td>
<td>1.36</td>
<td>13.81</td>
<td>40.25</td>
<td>59.5</td>
<td>40.25</td>
<td>34.25</td>
<td>0.244</td>
</tr>
<tr>
<td>5</td>
<td>0.5575</td>
<td>0.352</td>
<td>1.375</td>
<td>1.135</td>
<td>13.74</td>
<td>49.28</td>
<td>38.33</td>
<td>40.25</td>
<td>40.94</td>
<td>0.248</td>
</tr>
<tr>
<td>6</td>
<td>0.5575</td>
<td>0.352</td>
<td>1.0125</td>
<td>0.91</td>
<td>11.31</td>
<td>40.57</td>
<td>37.8</td>
<td>40.25</td>
<td>40.94</td>
<td>0.248</td>
</tr>
<tr>
<td>7</td>
<td>0.665</td>
<td>0.46</td>
<td>1.4625</td>
<td>1.36</td>
<td>24.81</td>
<td>74.62</td>
<td>59.5</td>
<td>59.5</td>
<td>59.5</td>
<td>0.347</td>
</tr>
<tr>
<td>8</td>
<td>0.665</td>
<td>0.46</td>
<td>1.375</td>
<td>1.135</td>
<td>21.12</td>
<td>63.52</td>
<td>59.5</td>
<td>59.5</td>
<td>59.5</td>
<td>0.347</td>
</tr>
<tr>
<td>9</td>
<td>0.665</td>
<td>0.46</td>
<td>1.0125</td>
<td>0.91</td>
<td>17.43</td>
<td>52.43</td>
<td>59.5</td>
<td>59.5</td>
<td>59.5</td>
<td>0.347</td>
</tr>
<tr>
<td>10</td>
<td>0.7825</td>
<td>0.5775</td>
<td>1.4625</td>
<td>1.36</td>
<td>36.56</td>
<td>93.45</td>
<td>59.5</td>
<td>59.5</td>
<td>59.5</td>
<td>0.347</td>
</tr>
<tr>
<td>11</td>
<td>0.7825</td>
<td>0.5775</td>
<td>1.375</td>
<td>1.135</td>
<td>31.18</td>
<td>79.69</td>
<td>59.5</td>
<td>59.5</td>
<td>59.5</td>
<td>0.347</td>
</tr>
<tr>
<td>12</td>
<td>0.7825</td>
<td>0.5775</td>
<td>1.0125</td>
<td>0.91</td>
<td>25.82</td>
<td>66.01</td>
<td>59.5</td>
<td>59.5</td>
<td>59.5</td>
<td>0.347</td>
</tr>
<tr>
<td>13</td>
<td>0.89</td>
<td>0.685</td>
<td>1.4625</td>
<td>1.36</td>
<td>49.46</td>
<td>111.14</td>
<td>59.5</td>
<td>59.5</td>
<td>59.5</td>
<td>0.347</td>
</tr>
<tr>
<td>14</td>
<td>0.89</td>
<td>0.685</td>
<td>1.375</td>
<td>1.135</td>
<td>42.4</td>
<td>95.3</td>
<td>59.5</td>
<td>59.5</td>
<td>59.5</td>
<td>0.347</td>
</tr>
<tr>
<td>15</td>
<td>0.89</td>
<td>0.685</td>
<td>1.0125</td>
<td>0.91</td>
<td>34.86</td>
<td>78.34</td>
<td>59.5</td>
<td>59.5</td>
<td>59.5</td>
<td>0.347</td>
</tr>
</tbody>
</table>

**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 the marginal variation he may calculate the section properties on the basis of an effective flange width of 1.33 m.
concrete bricks + joints + ribs
\[ B = (8 \times 215) + (7 \times 10) + (2 \times 1) \times 102.5 = 1.4625 \text{ m} \]

Rib spacing

\[ d = 0.255 \text{ m} \]
\[ D = 0.44 \text{ m} \]

Common depths

\[ d = 0.352 \text{ m} \]
\[ D = 0.575 \text{ m} \]

Figure 21: Typical bonded concrete brick diaphragm wall arrangements used for Table 2 data
2.4.2 Design bending moments

In order to design the required masonry 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 (MRs), maximum forces, moments and stresses will be found either at the base of the wall or at a level 3/8 \( h \) down from the top of the wall. If the MRs is less than the applied base wind moment of \( \gamma_W \cdot h^2/8 \), or significant lateral deflection of the roof prop occurs, the wall will tend to rotate and 'crack' at the base. As no tensile resistance is assumed at this level the design MRs does not decrease and any reduction in the lever arm of the vertical load due to rotation at the hinge is negligible and can be ignored. However, on the bending moment diagram, the level of the maximum wall moment will not now be at 3/8 \( h \) down from the top and its value will exceed \( 9\gamma_W \cdot h^2/128 \). The wall will become similar to a partially fixed ended beam. For example, suppose the value of a particular MRs is equivalent to, say, \( \gamma_W \cdot h^2/10 \), then the reactions at base and prop levels are:

- 0.6 \( \gamma_W \cdot h \) at base level
- 0.4 \( \gamma_W \cdot h \) at prop level (see Figure 26).

The stability moment of resistance chosen above is inadequate to resist the true propped cantilever base moment of \( \gamma_W \cdot h^2/8 \). The section is 'cracked' and any reserve of strength available at the higher level is called upon. This modifies the bending moment diagram from that of a true propped cantilever. The modification for the example under consideration is shown in Figure 27.

The applied wind moment at the level of 0.4 \( h \) down is calculated as:

\[
(0.4 \cdot \gamma_W \cdot h \times 0.4 \cdot h) - (0.4 \cdot \gamma_W \cdot h \times 0.2 \cdot h) = 0.08 \cdot \gamma_W \cdot h^2
\]

This exceeds the true propped cantilever wall moment of 0.07 \( \gamma_W \cdot h^2 \). The moment of resistance provided by the wall at this level must then be checked against the calculated maximum design bending moment.

In summary, the action of the wall may be 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 that of a true propped cantilever.

The initial assumption of a perfectly rigid prop generally provides the most onerous design condition. Considering the two locations of maximum design moment, it is apparent that the critical design condition invariably occurs at the higher location. The resistance at this location is dependent on the development of both flexural compressive and flexural tensile stresses.

2.4.3 Allowable flexural stresses

1. Allowable flexural tensile stress, \( f_{\text{ubt}} \):

   BS 5628: Part 1, Clause 36.4.3 gives:

   \[
   \text{design moment of resistance} = f_{\text{ubt}} Z/\gamma_m
   \]

   where \( f_{\text{ubt}} \) = characteristic flexural strength (BS 5628: Part 1, Clause 24)

   and \( \gamma_m \) = partial safety factor for materials (BS 5628: Part 1, Clause 27)

   For the purpose of this design guide, \( f_{\text{ubt}} / \gamma_m \) is termed allowable flexural tensile stress, \( f_{\text{ubt}} \).

2. Allowable flexural compressive stress, \( f_{\text{ubc}} \):

   BS 5628: Part 1 gives no consideration to flexural compressive stresses in designing laterally loaded elements although in the derivation of \( \beta \) (in Appendix B) the code discusses the application of a rectangular stress block of 1.1 \( f_{\text{ubc}} / \gamma_m \) to the resistance of bending moments produced by eccentric vertical loading. Consideration must also be given to the implication of the geometric form of the diaphragm...
2.4.4 Trial section coefficients $K_2$ and $Z$

The symmetrical profile of the diaphragm wall permits a simple method of arriving at a trial section. This considers the two critical conditions that exist in the ‘propped cantilever’ action of the analysis. Condition (1) occurs at the base of the wall – the applied bending moment at this level must not exceed the stability moment of resistance of the wall. Condition (2) occurs at approximately 3/8 $h$ down from the top of the wall – the flexural tensile stresses are a maximum at this level and must not exceed those allowable through calculation.

Consider the two conditions:

**Condition (1) at base of wall**

The trial section analysis is simplified by assuming that there is no flexural tension at this level and that the mass contributing to the $MR_s$ comprises only the self weight of the masonry.

\[
BM \text{ at base level } = \gamma_l W_s h^2/8
\]

\[
MR_s \text{ at base level } = \text{area} \times \text{height} \times p \times \gamma_l \times 0.475D = 0.475 (AhDp)
\]

Equating (1) and (2).

\[
\gamma_l W_s h^2/8 \leq 0.475 (AhDp)
\]

$\gamma_l$ for wind and dead loads will be taken as 1.4 and 0.9 respectively.

Hence $0.175 W_s h^2 \leq 0.4275 (AhDp)$

Let $K_2 = 0.4275 ADp$

Then $h \leq 5.714 K_2/W_s$

**Example:**

For the concrete blockwork diaphragm wall reference A in Section 2.3:

\[
K_2(15) = 0.4275 \times 0.2261 \times 0.435 \times 15
\]
= 0.631 kN per metre length of wall for blocks with a density of 1500 kg/m³

\[ K_3(20) = 0.4275 \times 0.2261 \times 0.435 \times 20 \]

= 0.841 kN per metre length of wall for blocks with a density of 2000 kg/m³

**Condition (2) at 3/8 h below top of wall**

BM at 3/8 h level = \( 9 \gamma_1 W_k h^3 / 128 \)  \( \text{(4)} \)

Moment of resistance = \( (f_{ax}/\gamma_m + g_o)Z \)  \( \text{(5)} \)

Equating (4) and (5).

\[ 9 \gamma_1 W_k h^3 / 128 \leq (f_{ax}/\gamma_m + g_o)Z \]

The trial section analysis is further simplified by assuming that flexural tensile stresses control; \( \gamma_m = 2.5; f_{ax} = 0.25 \) N/mm²; \( p \) for concrete blockwork is 1500 kg/m³.

By substitution,

\[ 9 \times 1.4 \times W_k h^3 / 128 \leq (0.25 \times 10^{3/2.5} + 0.9 \times 15 \times 3 / 8) Z \]

from which,

\[ Z = W_k h^3 / (1000 + 50.6 h) \]  \( \text{(6)} \)

Two graphs have been plotted for equations (3) and (6) and for various values of \( W_k \) (see Figures 28 and 29).

For a known wall height and wind pressure, values of \( K_3 \) and \( Z \) may be read from the graphs and, using Tables 1 or 2, 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 summarized thus:

(a) the wall acts as a true propped cantilever;
(b) there is no transfer of tension at the base;
(c) vertical roof loads (downward or uplift) are ignored;
(d) \( \gamma_m \) is taken as 2.5;
(e) \( f_{ax} \) is taken as 0.25 N/mm²;
(f) density of blockwork is taken as 1500 kg/m³;
(g) \( K_3 \) values are calculated using the approximate lever arm method.

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

**2.4.5 Shear stress coefficient \( K_3 \)**

It is also necessary to check the vertical shear stress at the junction of the cross-ribs and the leaves (see Figure 30).

![Figure 28: Graph to determine \( K_3 \)](image-url)

*Note: this trial section graph is based on the loading combination of dead plus wind for which the partial safety factors on loads are taken as 0.9 and 1.4 respectively.*
Vertical design shear stress \( v_n = \frac{VA_1\gamma t_i}{lb} \),

\( V = \) design shear force = \( \gamma x \) characteristic shear force

\( A_1 = B_d x t_i \) and \( \gamma = d/2 + t_i/2 \)

Then, \( v_n = V x B_d x t_i(d/2 + t_i/2)/lb \leq f/\gamma_{mv} \)

Generally, \( t_i = b \),

Therefore \( v_n = V x B_d(d/2 + t_i/2)/I \)

Now let \( K_1 = B_d(d/2 + t_i/2)/I \) = shear stress coefficient

Then \( v_n = VK_1 \)

Values of \( K_1 \) may be calculated for all diaphragm wall profiles and some are given in Tables 1 and 2.

Example:

For the concrete blockwork diaphragm wall reference A considered in Section 2.3.

\[ K_1 = 0.9/0.0053 (0.235/2 + 0.10/2) = 28.39 \text{ m}^{-2} \]

2.5 Vertical loading only

2.5.1 Slenderness ratio

Whilst in most cases of single-storey structures vertical loading is not critical, it is nevertheless advisable to follow where possible the recommendations of BS 5628 on this aspect. Thus it is necessary to assess the slenderness ratio of such walls, and to check that it does not exceed 27.

2.5.2 Effective height

There are difficulties in determining the effective height of diaphragm walls. If a wall is considered as a propped cantilever, it would be reasonable to suggest that the effective height is 0.875 times the actual height. Under the action of wind pressure on the wall and suction on the roof, however, the restraining action of the prop could be reduced and the effective height could be greater than this figure. The assessment of the effective height must therefore be judged by the designer for each individual case.

![Graph to determine Z](image-url)

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

Figure 29: Graph to determine Z
2.5.3 Effective thickness

BS 5628: Part 1 does not provide a satisfactory means of determining the effective thickness of diaphragm walls. It might be considered that the effective thickness should be assessed as two-thirds of the sum of the thicknesses of the two leaves, i.e. the same as a normal cavity wall. This, however, does not take account of the composite box action of the section. In most structural codes slenderness ratio is related to radius of gyration, but in BS 5628: Part 1 slenderness ratio is defined as the ratio of effective height to effective thickness. This is because this code takes account of only plane solid and cavity walls and the radius of gyration of such walls is directly proportional to their thickness. For geometric walls, such as the diaphragm, this is clearly not the case and it would be preferable if the radius of gyration were to be used for all walls.

For the purposes of this design guide the effective thickness of a diaphragm wall is taken to be the overall thickness. This gives a conservative design solution as explained in the following:

Consider the diaphragm shown in Figure 31 in which the overall thickness is 550 mm, \( I = 12.5 \times 10^{-3} \text{ m}^4 \) and \( A = 0.271 \text{ m}^2 \).

\[
A = 1 \times t
\]

Radius of gyration \( r = \sqrt{IA} = \sqrt{12.5 \times 10^{-3}/0.271} = 0.215 \text{ m} \)

Thus, for the same radius of gyration, the solid wall requires a thickness of

\[
t = 0.215 \times 3.46 = 744 \text{ mm}
\]

Hence the diaphragm wall has considerable inbuilt stiffness since 550 mm overall thickness of diaphragm wall has an equivalent slenderness ratio \( (I/r) \) to a solid wall of 744 mm thickness, and the rule 'effective thickness of diaphragm wall equals overall thickness' is seen to be on the conservative side.

2.5.4 Eccentricity of vertical loading

The eccentricity of the vertical loading depends upon the means of application of the load.

The majority of diaphragm walls are capped by reinforced concrete ring beams, to which the roofs are bolted. If the roofs were not to deflect, there would be zero slope of the roof members at their connection to the capping beam. Such a theoretical condition does not arise, for the roofs will deflect, even under their own weight. There is then a slope of the roof members at the support, resulting in the roof/wall contact being eccentric. In the extreme case, the contact could be close to the inner face of the inner leaf (Figure 32).
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. However, this dispersion is unlikely to be sufficient to cause the outer and inner leaves to be loaded equally. The inner leaf will be more heavily stressed than the outer leaf. For bonded cross-ribs this could be considered as a local bearing stress, since the roof beam loads are applied at intervals. Clause 34 of BS 5628: Part 1 allows up to 50% increase in the local stress. The rib also probably disperses the excess stress from the inner to the outer leaf. For tied cross-ribs, the designer should assess the effect of the eccentricity on an individual job basis, taking account of the magnitude of the vertical loading and the contribution of the shear ties in transferring the load from inner to outer leaves.

The problem of eccentricity can often be more economically controlled by detailing the bearing of the roof beams on the capping beams so that the load is applied where the designer wants it (Figure 33).

2.5.5 Capacity reduction factor $\beta$

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 of BS 5628: Part 1 using the actual thickness as the effective thickness.

3 Design procedure and worked examples

A suggested procedure for the design of a diaphragm wall in an open-plan single-storey building is as follows:

- **Stage 1**: Calculate the characteristic and design loads from dead, superimposed and wind loading on the structure.
- **Stage 2**: Select a trial section using Figures 28 and 29.
- **Stage 3**: Calculate size of ring beam if required.
- **Stage 4**: Assume a block strength (or brick strength) and mortar designation.
- **Stage 5**: Check the external leaf spanning between cross-ribs for adequate thickness.
- **Stage 6**: Calculate the design wind moment and the stability moment of resistance at the base of the wall.
- **Stage 7**: Calculate the design flexural stresses at the location of maximum applied moment in the wall, $M_w$.
- **Stage 8**: Calculate the flexural resistance of the masonry at the level of maximum $M_w$ and compare it with the design stresses calculated in stage 7.
- **Stage 9**: Calculate the shear stress in the cross-ribs.
- **Stage 10**: Design the shear resistance of the cross-ribs.
- **Stage 11**: Summarize conditions (a) (b) (c) and (d) for cross-rib spacing (see Section 2.2.1).
- **Stage 12**: Design the roof plate and transverse shear walls (these aspects of the design are outside the scope of this design guide).
- **Stage 13**: Check the loading combinations of (a) dead plus imposed plus wind and (b) dead plus imposed only.
Worked example 1: Warehouse building
The warehouse measures 40 x 75 x 7.5 m high and is shown in Figure 34. It is to be designed in concrete blockwork, using diaphragm wall construction for its main vertical structure. There are no internal walls to provide intermediate support to the external walls. Normal testing of materials and minimum supervision of the workmanship will be employed during the building programme.
Solid concrete facing blocks with a compressive strength of 7.0 N/mm² will be used throughout the building, and are assumed to have a density of 2000 kg/m³ (20 kN/m³).

Stage 1. Loading
(a) Wind loads
The basic wind pressure on any building is calculated in accordance with the relevant British Standard (CP 3, Chapter V, Part II).
For this design example the basic pressure and local pressure intensities can be assumed to have been calculated as:
Dynamic wind pressure, $q = 0.65$ kN/m²
$C_{pw}$ on windward face $= 0.70$
$C_{pw}$ on leeward face $= 0.60$
$C_{p}$ on roof $= 0.80$
$C_{p}$ on walls $= \text{either } +0.2 \text{ or } -0.3$

Therefore, characteristic value wind loads are:
Pressure on windward wall $= W_{w1} = (C_{pw} - C_{p})q = (0.70 - 0.30)0.65 = 0.65$ kN/m²
Suction on leeward wall $= W_{w2} = (C_{pw} - C_{p})q = (0.60 - 0.20)0.65 = 0.26$ kN/m²
Gross wind uplift on roof $= W_{w3} = (C_{pw} + C_{p})q = (0.8 + 0.2)0.65 = 0.65$ kN/m²

(b) Dead and superimposed loads
Characteristic superimposed load $Q_{k} = 0.75$ kN/m² (no general access to roof)

Characteristic dead load $G_{k}$ assumed to be
metal decking and insulation $= 0.165$
felt and chippings $= 0.285$
self weight of roof beams $= 0.200$
Total $G_{k} = 0.65$ kN/m²

(c) Design loading
The critical loading for tall single-storey walls is usually dead + wind although the loading condition of dead + imposed + wind should also be checked.
Design dead load $= 0.9G_{k}$ or $1.4G_{k}$
Design wind load $= 1.4W_{k}$ or $0.015G_{k}$

By inspection, the most critical loading combination will be given by:
Design dead load $= 0.9 \times 0.65 = 0.585$ kN/m²
Design wind loads:
pressure $W_{k1} = 1.4 \times 0.65 = 0.91$ kN/m²
suction $W_{k2} = 1.4 \times 0.26 = 0.364$ kN/m²
uplift $W_{k3} = 1.4 \times 0.65 = 0.91$ kN/m²
Resultant load on roof $= 0.585 - 0.91 = -0.325$ kN/m² (i.e. uplift)

Stage 2. Select trial section
For a wall height of 7.5 m and a characteristic wind load of 0.65 kN/m², trial section coefficients $K_{1} = 0.85$ kN/m and $Z = 27.5 \times 10^{-3}$ m are read from Figures 28 and 29 respectively.
Although Section D from Table 1 meets the requirements for $K_{2}$ and $Z$, it is less economic than Section E. Therefore select wall section E and analyse this section fully. The wall section to be analysed is shown in Figure 31 and comprises 550 mm overall thickness with bonded cross-ribs spaced at 1180 mm centres.
As noted earlier, in practice the bonded wall may require rendering to provide adequate resistance to rain penetration.
The properties of this wall section have been calculated as:
$I$ per metre $= 0.0106$ m⁴
$Z$ per metre $= 0.038$ m³
$A$ per metre $= 0.230$ m²
$K_{3}$ (20) $= 1.080$ kN/m height
$K_{1}$ $= 21.24$ m⁻²

Stage 3. Capping beam
The design wind uplift in this example exceeds the dead load of the roof.
The main roof beams are spaced at 4.8 m centres and the uplift reaction at each roof beam bearing
$= (4.8 \times 0.325)40/2 = 31.20$ kN.
The roof beam will be anchored to a continuous reinforced concrete capping beam, as shown in Figure...
35, which will assist in counterbalancing the nett uplift reaction from the roof beams.

Dead load counterbalance from reinforced capping beam
\[ \text{beam} = \text{area} \times \text{length} \times \text{density} \times \gamma_y \]
\[ = 0.60 \times 0.55 \times 4.8 \times 24 \times 0.9 = 34.20 \text{ kN} \]
(OK: 31.20 kN uplift).

Hence the capping beam is adequate to counterbalance the roof beam uplift force. Alternatively, a shallower capping beam could be used and anchorage made into the masonry to provide the required counterbalance mass.

If a capping beam is not to be used, the wind uplift force may be resisted by means of metal straps built into the masonry using the dead load of the masonry as the uplift counterbalance. The designer must allow for the effect of the reduced dead load, resulting from this uplift force, in the stress calculations for bending across the wall section and in the calculation of the stability moment of resistance at the base of the wall.

Stage 4. Specification of blocks and mortar

The blocks to be used throughout have been specified as having a compressive strength of 7.0 N/mm² and a density of 2000 kg/m³ and are to be set in a designation (iii) mortar (1 : 1 : 6). The work size of the blocks to be used is 440 x 215 x 100 mm.

Stage 5. Check external leaf spanning between cross-ribs

Condition (a). Section 2.2.1, for checking the spacing of the cross-ribs relates to the leaf acting as a continuous slab spanning between the cross-ribs to support the lateral loading, in this case, wind.

\[ M = \gamma_y W_e B_r^2 / 10 \]
\[ = 1.4 \times 0.65 \times 1.18^2 / 10 \]
\[ = 0.127 \text{ kNm} \]

Design moment of resistance, \( M_R = f_{cx} Z / \gamma_m \)

For the mortar specified,
\[ f_{cx} = 0.6 \text{ N/mm}^2 \text{ (7.0 N/mm}^2 \text{ blocks in designation (iii) mortar with plane of failure perpendicular to bed joint)} \]
\[ \gamma_m = 3.5 \text{ (from Table 4 of BS 5628: Part 1, for normal control of both construction and manufacture of structural units)} \]
\[ Z = 1 \times t_r^2 / 6 = 1 \times 1.0^2 / 6 = 0.00167 \text{ m}^3 \]

hence \( MR = (0.6/3) \times 0.00167 \times 10^4 = 0.286 \text{ kNm which is greater than the applied bending moment of 0.127 kNm} \)

Stage 6. Design wind moment and \( M_{R_1} \), at base of wall

Consider a 1 m length of wall:

Design wind moment at base = \( \gamma_y W_e / h^2 / 8 \]
\[ = 1.4 \times 0.65 \times 7.5^2 / 8 \]
\[ = 6.40 \text{ kNm} \]

Figure 36. Roof anchorage to reinforced concrete capping beam

Stability moment of resistance = axial load x lever arm

The axial load for this design example comprises only the design dead load of the masonry (as the wind uplift cancels out the dead loading from the roof and the capping beam) and is calculated as:
\[ \gamma_y \times \text{area} \times \text{density} \times \text{height} = 0.9 \times 0.23 \times 20 \times 7.5 \]
\[ = 31.05 \text{ kN} \]

The lever arm of the axial load (see Figure 24) is calculated by first establishing the minimum width of the stress block at the point of rotation.

Minimum stress block width \( w_s = \text{axial load} / P_{ubc} \)
where \( P_{ubc} = \text{allowable flexural compressive stress} \)

Concrete blocks with a compressive strength of 7.0 N/mm² set in a designation (iii) mortar have been specified. Therefore, from Table 2d of BS 5628: Part 1, \( f_c = 6.4 \text{ N/mm}^2 \). The foundation is assumed to comprise a reinforced raft, and a section through the edge beam is shown in Figure 36. The raft foundation affords full restraint to the wall at this level and hence \( \beta \) may be taken to be 1.0.

Hence, \( P_{ubc} = 1.1 \times 6.4 / 3.5 = 2.01 \text{ N/mm}^2 \)

Now, assuming the stress block to be within the leaf thickness, the minimum width of stress block, \( w_s \), is given by
\[ w_s = 31.05 \times 10^3 / (1000 \times 2.01) \]
\[ = 15.45 \text{ mm} \]
(i.e. assumption correct – within leaf thickness)

Hence lever arm = wall thickness/2 – \( w_s / 2 \)
\[ = 550/2 – 15.45/2 = 267 \text{ mm} \]

and stability moment of resistance, \( M_{R_1} \)
\[ = 31.05 \times 0.287 \]
\[ = 8.29 \text{ kNm (see Figure 37)} \]

This is greater than the applied design wind moment, at the base of the wall, calculated earlier as 6.40 kNm.

Stage 7. Design flexural stresses

Since the stability moment of resistance at the base of the wall 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 height is, therefore, located at 3/8 \( h \) down from the roof prop.
The design axial load again comprises only the design self weight of the masonry; therefore, at the level of $M_w$:

$$
\text{design axial load } = \gamma_f \times \text{area} \times p \times 3/8 \times \text{height} \\
= 0.9 \times 0.23 \times 20 \times 3/8 \times 7.5 \\
= 11.644 \text{ kN}
$$

Then, from load/area/moment/section modulus, the design flexural tensile stress ($f_{ubt}$) and the design flexural compressive stress are calculated as follows:

$$
\begin{align*}
    f_{ubt} &= \frac{(11.644 \times 10^3) / (0.23 \times 10^6) - (3.6 \times 10^6) / (38.0 \times 10^6)}{0.0506 - 0.0947} \\
    &\quad = -0.0441 \text{ N/mm}^2 \\
    f_{ucb} &= 0.0506 + 0.0947 = 0.1453 \text{ N/mm}^2 \text{ (see Figure 39)}
\end{align*}
$$

**Stage 8. Allowable flexural stresses at level of $M_w$**

(a) **Allowable flexural tensile stress**

$$
P_{ubt} = \frac{f_{ux}}{\gamma_m}
$$

- $f_{ux} = 0.25 \text{ N/mm}^2$ for concrete blocks with a compressive strength of 7 N/mm$^2$ set in a designation (iii) mortar, from Table 3 of BS 5628: Part 1, for the plane of failure parallel to the bed joints.
- $\gamma_m = 3.5$
- $P_{ubt} = 0.25 / 3.5 = 0.071 \text{ N/mm}^2$

This is greater than the applied $f_{ubt} = 0.0441 \text{ N/mm}^2$; the flexural tensile stresses are therefore acceptable.

(b) **Allowable flexural compressive stress**

$$
P_{ucb} = 1.1 \beta_f / \gamma_m
$$

- $f_k = 6.4 \text{ N/mm}^2$ for blocks with a compressive strength of 7 N/mm$^2$ set in a designation (iii) mortar (interpolated from Table 2d of BS 5628: Part 1)
\( \gamma_m = 3.5 \)

The effective length of the flanges 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, \( SR = 0.75 \times 1.080/0.10 = 8.10 \)

The stressed area is trapezoidal in shape but its centroid is unlikely to fall outside 0.05 of the flange thickness as an eccentricity. At this stage of the design the eccentricity cannot be accurately computed as the stress value has not yet been determined. To simplify the calculation, an eccentricity of 0.1\( t_e \) will be catered for. Therefore, with \( SR = 8.10 \) and \( e_e = 0.1t_e \) from BS 5628 : Part 1, Table 7, \( \beta = 0.88 \).

\[
P_{obc} = 1.1 \beta f_e / \gamma_m = 1.1 \times 0.88 \times 6.4 / 3.5 = 1.77 \text{ N/mm}^2
\]

which is greater than the applied \( f_{obc} = 0.1453 \text{ N/mm}^2 \); thus the flexural compressive stresses are also acceptable.

**Stage 9. Shear stresses in cross-ribs**

Horizontal reaction at base = design shear force \( V \)

\[
V = 5/8 \times \gamma_i \times W_4 \times h \\
= 5/8 \times 1.4 \times 0.65 \times 7.5 \\
= 4.266 \text{ kN/m}
\]

or \( = 4.266 \times 1.180 = 5.034 \text{ kN per cross-rib} \)

Design shear stress at cross-rib/leaf interface = \( \nu_h \)

\[
\nu_h = K_i V \\
= 21.24 \times 5.034 / 10^3 \\
= 0.107 \text{ N/mm}^2
\]

The maximum shear stress on the bedjoints of the cross-ribs is shown in Figure 40 and is calculated from

\[
\nu_h = V A_2 \bar{y} / Ib_i
\]

\[
A_2 = \text{area of hatched portion of wall in Figure 40} \\
= (1.18 \times 0.1) + (0.175 \times 0.1) = 0.1355 \text{ m}^2
\]

\[
\bar{y} = \text{distance from centre line of wall to centroid of area } A_2 \\
= [(1.18 \times 0.1 \times 0.225) + (0.175 \times 0.1 \times 0.0875)] / 0.1355 = 0.207 \text{ m}
\]

\[
\nu_h = (5.034 \times 0.1355 \times 0.207) / (0.0125 \times 0.1 \times 10^3) \\
= 0.113 \text{ N/mm}^2
\]

**Stage 10. Shear resistance of cross-ribs**

The characteristic shear strength of masonry, \( f_s \), is given in BS 5628 : Part 1, Clause 25 as \( f_s = 0.35 + 0.6 g_k \) for mortar designation (iii).

This relates to the shear stress in the bed joint and includes the benefit of the vertical load in the wall.

Hence design shear strength = \( (0.35 + 0.6 g_k) / \gamma_m \)

where \( g_k = \text{design vertical load per unit area} \\
= \gamma_i \times A \times h / A = 0.9 \times 0.271 \times 20 \times 7.5 / (0.271 \times 10^3) = 0.135 \text{ N/mm}^2 \)

therefore design shear strength = \( [0.35 + 0.6 \times 0.135] / 3.5 \)

\( = 0.123 \text{ N/mm}^2 \)

which is greater than the maximum shear stress calculated earlier as 0.113 N/mm².

BS 5628 : Part 1 does not give shear strengths of blocks subjected to the mode of failure at the bonded vertical interface of cross-rib to leaf in which the blocks themselves, as well as the vertical mortar joints in alternate courses, are in shear.

Until the results of research into this matter are available the shear strength of concrete masonry must be estimated. It is evident that its shear strength will be considerably greater than that of the mortar joints alone (calculated above as 0.123 N/mm² for this example). However, the designer must also consider the implications of the bonding arrangement and the

---

![Maximum shear stress in the bed joints of the cross-ribs](image-url)

![Design flexural stresses](image-url)

Figure 40: Maximum shear stress in the bed joints of the cross-ribs

---

![Shear stress distribution](image-url)

maximum shear stress in cross-rib

---

25
effectiveness of the perpendicular mortar joints at the cross-rib/leaf interface.

It is recommended that where the vertical shear resistance is to be provided by bonded masonry, only units with a minimum compressive strength of 7 N/mm² should generally be used. The use of less dense units in this bonded form of construction should be avoided until research into shear reaches a conclusion or adequate test information is provided by the designer. Several design alternatives are available including the use of tied cross-ribs with designed metal shear ties, reducing the spacing of the cross-ribs or increasing the spacing of the leaves. Each of these latter two options reduces the shear stresses on the cross-ribs.

Stage 11. Summarize cross-rib spacing conditions (see Section 2.2.1)
Condition (a) – satisfactory (see stage 5).

Condition (b) \( B_{\text{max}} = 27 \times t_r \)
\[ = 27 \times 0.10 \]
\[ = 2.70 \text{ m} \]

This is greater than the breadth adopted for the trial design, 1.180 m (see stage 2).

Condition (c) \( B = 6t_r + 6b_r + 6t_r \) (but \( t_r = b_r \))
\[ : B = 0.10 \times 13 \]
\[ = 1.30 \text{ m} \text{ which is greater than} \]
\[ 1.180 \text{ m} \]

or
\[ B = h/3 \]
\[ = 2.50 \text{ m}, \text{ which is greater than} \]
\[ 1.180 \text{ m} \]

Condition (d) was shown to be acceptable in stages 9 to 10.

Stage 12. Roof plate and shear walls
The design of the roof plate and transverse shear walls is outside the scope of this design guide.

Stage 13. Check loading combinations 'dead + superimposed + wind', and 'dead + superimposed'
The most crucial design condition, as stated earlier, is generally that of dead plus wind loading. The flexural tensile stresses are likely to be the limiting factor in the design, as was shown by calculation.

The designer should now check the 'dead + superimposed + wind' loading combination in which the flexural compressive stresses will be greater than those for the 'dead + wind loading' combination. It is expected that they will remain comfortably within the allowable values calculated. The design assumes that the roof beam bearing is detailed such that its loading is applied on the centre-line of the diaphragm wall section.

Figure 41: Design bending moment diagram for combined loading

'Dead + superimposed + wind'

Design loads
Dead + superimposed + wind = 1.2 \( G_k + 1.2 \) \( Q_k + 1.2 \) \( W_k \)

Roof dead load = 1.2 \( 0.65 = 0.78 \text{ kN/m}^2 \)

Roof superimposed load = 1.2 \( 0.75 = 0.90 \text{ kN/m}^2 \)

Wind loading on walls = 1.2 \( 0.65 = 0.78 \text{ kN/m}^2 \)

Wind uplift on roof = 1.2 \( 0.65 = 0.78 \text{ kN/m}^2 \)

(Dead + superimposed load is greater than wind uplift)

Base wind moment
\( M_{\text{base}} = \gamma_W W_r h^2/8 = 0.78 \times 7.5^2/8 = 5.48 \text{ kNm} \)

This is less than that for the 'dead + wind' loading combination previously calculated. The stability moment of resistance, allowing for the additional vertical loading, will be greater than previously calculated. Thus, the wall will again be considered to act as a true propped cantilever for this loading combination.

Wall wind moment
\( M_w = 9\gamma_W W_r h^2/128 = 9 \times 0.78 \times 7.5^2/128 = 3.085 \text{ kNm} \)

The design bending moment diagram for this loading combination is shown in Figure 41.

Stresses at level of \( M_w \)

Design load + superimposed at 3/8 \( h \) (40 m span of roof beam)

- roof dead load = 0.78 \( 40/2 = 15.6 \)
- superimposed load = 0.90 \( 40/2 = 18.0 \)
- self-weight of masonry = 1.2 \( 0.23 \times 20 \times 3/8 \times 7.5 = 15.5 \)

Total = 49.1 \text{ kN/m}
Then flexural compressive stress, \( f_{\text{fc}} \)
\[
= (49.1 \times 10^3)/(0.23 \times 10^3) \\
+ (3.085 \times 10^3)/(38.0 \times 10^3) \\
= 0.213 + 0.081 \\
= +0.294 \text{ N/mm}^2
\]

and flexural tensile stress, \( f_{\text{ft}} \)
\[
= 0.213 - 0.081 \\
= +0.132 \text{ N/mm}^2, \text{ i.e. compressive}
\]

Thus compressive stress covers the full width of the section, as shown in Figure 42.

The maximum stress is within the previously calculated allowable value.

'Dead + superimposed'

A check should also be made on the overall stability of the wall and the associated maximum axial compressive stresses.

**Design loads**

Dead + superimposed \( = 1.4 Q_e + 1.60 Q_s \)

Roof dead load \( = 1.4 \times 0.65 = 0.91 \text{ kN/m}^2 \)

Roof superimposed load \( = 1.6 \times 0.75 = 1.20 \text{ kN/m}^2 \)

Self-weight of masonry per metre height \( = 1.4 \times 0.23 \times 20 = 6.44 \text{ kN/m} \)

Total design axial load (at base of wall)

\[
\begin{align*}
\text{roof dead load} & = 0.91 \times 40/2 = 18.20 \\
\text{roof superimposed load} & = 1.20 \times 40/2 = 24.00 \\
\text{self-weight of masonry} & = 6.44 \times 7.5 = 48.30 \\
\text{Total} & = 90.50 \text{ kN/m}
\end{align*}
\]

Total design axial load (at midheight of wall)

\[
\begin{align*}
\text{roof dead + superimposed load} & = 18.2 + 24.0 = 42.40 \\
\text{self-weight of masonry} & = 6.44 \times 3.75 = 24.15 \\
\text{Total} & = 66.55 \text{ kN/m}
\end{align*}
\]

**Capacity reduction factor**

Eccentricity of loading, \( e = 0 \)

Slenderness ratio, \( SR = 0.75 \times 7.5/0.55 = 10.23 \)

Then, from BS 5628 : Part 1, Table 7 with \( e_x = 0 \) and \( SR = 10.23 \)

\( \beta = 0.965 \) (by interpolation)

Therefore, design vertical load resistance

\[
\beta \times \text{area} \times f_y \gamma_m \\
= 0.965 \times 0.23 \times 6.4 \times 10^3/3.5 \\
= 406 \text{ kN/m}
\]

This exceeds the total design axial load calculated at 66.35 kN/m and demonstrates that in practice it is usual for only the dead + wind load combination to be designed.

Maximum axial compressive stress (at base of wall)
\[
= (66.55 \times 10^3)/(0.23 \times 10^3) \\
= 0.288 \text{ N/mm}^2
\]

Maximum allowable axial compressive stress (i.e. with no slenderness reduction)

\[
= 6.4/3.5 \\
= 1.83 \text{ N/mm}^2
\]

**Worked example 2: Retaining wall in unreinforced masonry (uncracked section)**

A retaining wall is required to support a non-cohesive granular fill to a height of 1.5 m. The surface of the fill is level as shown in Figure 43 and for this example there are no requirements for surcharge loading. A dpc consisting of two courses of slate bedded in mortar is to be used. The design stages are varied from the previous example to meet the special requirements of the problem.

**Stage 1. Calculate lateral loading, overturning moment and shear**

Lateral earth pressure \( = K_s \rho h \)

where \( K_s \) is coefficient of active earth pressure and \( \rho \) is bulk density of soil

Design lateral earth pressure \( = \gamma_r K_s \rho h \)

\[
\gamma_r = 1.6 \\
K_s = \frac{1 - \sin \theta}{1 + \sin \theta} = 0.27 \\
\rho = 15 \text{ kN/m}^3
\]

Consider 1 m length of wall

Design lateral earth pressure at base
\[
= 1.6 \times 0.27 \times 15 \times 1.5 \\
= 9.72 \text{ kN/m}^2
\]
Design shear at base, \( V = 9.72 \times 1.5/2 \)
\[ = 7.29 \text{kN} \]

Design moment at base, \( M = 7.29 \times 1.5/3 \)
\[ = 3.65 \text{kNm} \]

**Stage 2. Choose unit strength, unit size and mortar grade**

Assume solid concrete blocks are used with a material density of 20 kN/m\(^3\) and a compressive strength of 7 N/mm\(^2\). The face size of the blocks to be used is 440 x 215 x 100 mm. The blocks are laid in mortar designation (iii). The blocks are to be manufactured with special category control of manufacture. Materials testing and full time site supervision of workmanship will be undertaken.

**Stage 3. Calculate section modulus and select trial section**

A slate dpc is to be used. This must be mortar bedded and the construction will be fully supervised, enabling flexural tensile stress to be considered at the base.

From BS 5628, Table 3, the characteristic flexural strength \( f_{ck} \) of concrete blocks of 7 N/mm\(^2\) in mortar designation (iii) with the plane of failure parallel to the bed joints is 0.25 N/mm\(^2\).

From BS 5628, Table 4, the partial safety factor on material strength with 'special/special' quality control \( \gamma_m = 2.5 \).

Design flexural strength \( = \frac{f_{ck}}{\gamma_m} = 0.25/2.5 \)
\[ = 0.1 \text{N/mm}^2 \]

Where tensile stress can be developed the stress diagram at the base will be as shown in Figure 44.

Stress due to the self-weight of the wall \( P/A \)
\[ = \gamma_p W/A \]
\[ = 0.9 \times 20 \times 1.5 \times 4 \times 10^3/(4 \times 10^6) \]
\[ = 0.027 \text{N/mm}^2 \]

To ensure that the design flexural strength is not exceeded

\[ P/A - M/Z = \gamma_m f_{ck} \]
\[ Z \text{ required} = 3.65 \times 10^3/0.127 \]
\[ = 29 \times 10^{-3} \text{m}^3/\text{per m length} \]

From Table 1, Section D provides the \( Z \) required so this section will check.

**Stage 4. Check maximum rib spacing**

With the increased lateral loading on the flange in a retaining wall the ability of the flange to span between ribs generally governs the rib spacing.

At the base of the wall the lowest section of flange will span two ways (see Figure 45). This is taken into account to provide the maximum economy of design.

Design pressure at base
\[ = 9.72 \text{kN/m}^2 \]

Design pressure at 315 mm above base
\[ = 1.6 \times 0.27 \times 15 \times (1.5 - 0.315) \]
\[ = 7.68 \text{kN/m}^2 \]

Average design pressure on triangular base
\[ = (9.72 + 7.68)/2 \]
\[ = 8.7 \text{kN/m}^2 \]

Total design load on triangle
\[ = 8.7 \times 0.63 \times 0.315/2 \]
\[ = 0.86 \text{kN} \]

Design moment at base
\[ = 0.86 \times 0.315/3 \]
\[ = 0.09 \text{kNm} \]

Design stress, \( M/Z = 0.09 \times 10^{10}/(630 \times 10^2) \)
\[ = 0.086 \text{N/mm}^2 \]

Design flexural strength parallel to bed joint
\[ f_{ck,per}/\gamma_m = 0.1 \text{N/mm}^2 \]

i.e. \( M/Z = f_{ck,per}/\gamma_m \)

Design bending moment on plane perpendicular to bed joint
\[ = \gamma_m Wb_e^2/10 \]

Consider 1 m height of wall

Design loading, \( \gamma_m W = 7.68 \text{kN/m} \)

Design moment = 7.68 x 0.73/10
\[ = 0.41 \text{kNm} \]

Design stress perpendicular to bed joints
\[ M/Z = 0.41 \times 10^{10} \times 6/(10^3 \times 10^2) \]
\[ = 0.246 \text{N/mm}^2 \]

Design flexural strength perpendicular to bed joints
\[ f_{ck}/\gamma_m \]
\[ = 0.6/2.5 \]
\[ = 0.24 \text{N/mm}^2 \]

Acceptable, as the loading reduces with height.

**Stage 5. Check horizontal applied shear stress**

Horizontal design shear force, \( V = 7.29 \text{kN per m length} \)

Shear force per diaphragm = 7.29 x 0.73
\[ = 5.32 \text{kN} \]

Horizontal shear stress, \( v_e = VA_e/1B \)

\( A_e \) (see Figure 40) = 0.73 x 0.1 + 0.1 x 0.35/2 = 0.0905 m\(^2\)

\[ \bar{V} = \frac{0.73 \times 0.1(0.35/2 + 0.1/2) \times (0.1 \times 0.35/2 \times 0.35/4)}{0.0905} \]
\[ = 0.198 \text{m} \]

From Table 1, \( I = 7.87 \times 10^{-3} \text{m}^4 \)

\[ v_e = \frac{5.32 \times 10^{-3} \times 0.0905 \times 10^6 \times 0.198 \times 10^{-3}}{7.87 \times 10^{-3} \times 10^{12} \times 100} \]
\[ = 0.121 \text{N/mm}^2 \]
Design shear strength \[ f_v / f_y = (0.35 + 0.6 g_x) / f_y \]
where \( g_x \) = Design vertical stress as previously calculated
\[ = 0.027 \text{ N/mm}^2 \]

Design shear strength
\[ [0.35 + (0.6 \times 0.027)] / 2.5 \]
\[ = 0.146 \text{ N/mm}^2 > v_n \]
(satisfactory)

**Stage 6. Calculate applied vertical shear stress at rib/flange interface**

Check the shear stress at the rib/flange interface.

Vertical design shear force \( V' = 5.32 \text{ kN per diaphragm} \)

Vertical shear stress \( v_r = K_1 V \) (see Table 1 for \( K_1 \))
\[ = 20.87 \times 5.32 \times 10^3 \text{ N/mm}^2 \]
\[ = 0.111 \text{ N/mm}^2 \]

Shear strength of mortar joints only:
\[ f_v / f_y = 0.35 / 2.5 = 0.14 \text{ N/mm}^2 \] (satisfactory)

**Stage 7. Check compressive stress**

Applied compressive stress (see Figure 44)
\[ = P / A + M / Z \]
\[ = 0.027 + 3.65 \times 10^5 / (39.2 \times 10^6) \]
\[ = 0.12 \text{ N/mm}^2 \]

**Figure 45. Flange restraint at the base of the wall**

Height of block/least horizontal dimension
\[ = 215 / 100 = 2.15 \]
Therefore, from BS 5628, Table 2(d):
At the base, \( \beta = 1 \) (i.e. full restraint)
Design compressive strength
\[ = f_v / f_y = 6.4 / 2.5 \]
\[ = 2.56 \text{ N/mm}^2 \]
This is to be expected when a design is based on tensile stress; the applied compressive stress is much less than the design strength.

**4 Bibliography**