LOADBEARING BRICKWORK CROSSWALL CONSTRUCTION
This is one of the tallest half-brick slender crosswall structures in Europe.

There were two main restrictions on this project: site area and construction costs. The cost of the hospital had already exceeded the budget, and this block had to be a no-frills building. Loadbearing brickwork was chosen for the optimum economy and reliability.

The structural design uses half-brick (102.5 mm) internal loadbearing walls throughout, except where sound or fire regulations demanded thicker walls, eg, around staircases and lift shafts. External walls are simple cavity walls with half-brick thick leaves. Floor slabs are solid reinforced concrete, 150 mm thick, partially precast to minimise shuttering operations on site, and comprise 65 mm thick prestressed planks with an 85 mm thick insitu topping.

The maximum designed masonry strength in the lowest storeys required bricks with a crushing strength of 50 N mm$^2$ set in a designation (ii) (1:3:4:) mortar.

To achieve a satisfactory compromise between economy and unnecessary and counter-productive confusion for the contractor, the masonry specification was reduced at three levels in the height of the structure. The bottom three storeys were constructed in engineering bricks, the top storeys in commons, and the intermediate floors in medium strength bricks.

For construction purposes, the building was divided into two halves. At each level, when the bricklayers had completed one half, they moved into the other while the concrete floors were constructed in the first half. In this way, continuity of work was maintained at all times for the relatively small work force.

Architects William Holford & Associates
Structural engineers W. G. Curtin & Partners.
# Loadbearing brickwork crosswall construction

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The Brick Development Association
This was the job (1958) that was to herald a new era in structural brickwork design. The original design was for a steel frame with 4in breeze block classroom separating walls. For acoustic reasons, the client then changed the brief from 4in breeze block partitions to 9in brickwork. This meant massively heavier loads on beams and columns. Thus, all the steelwork sections would have to be increased in size – and cost.

The first cost-saving solution was to pin each floor lift of brickwork tightly up against the soffit of the steel beams over, so that, in effect, the 4-storey height walls would be virtually self-supporting and not impose extra loads on the steel frame. The stress in the brickwork, due to its self-weight, at the base of the walls was checked and found to be insignificant.

A check was then made to determine whether the walls could possibly carry the 7.5 m spans of floors and roof. They could – and the structural steel frame was redundant.

Engineering bricks were used at the projecting ends of the crosswalls to prevent damp penetration – a vertical dpc being ruled out because of the danger of the projecting ends peeling off from the internal crosswalls.

The final surprise to the structural designers was to discover that as compared with reinforced concrete and steel-framed structures, the brickwork solution was not only cheaper but considerably faster to build.

Architects Weightman & Bullen
Structural engineers W. G. Curtin & Partners.
more than adequate to provide longitudinal stability, which is discussed in more detail later.

For office buildings where the room functions are accurately known in advance, the crosswall centres can be predetermined. Where greater flexibility is required in some areas, it is often possible to span the floor in the opposite direction onto the corridor and external walls for that area of the layout, and to introduce demountable partitions to suit requirements. However, where maximum flexibility is required, the crosswall form of construction is more restrictive than the spine wall form, where the floors span between external and corridor (or spine) walls throughout. This latter form of construction will be the subject of a future BDA Design Guide.

In many cases, the long floor spans are most economically formed in precast prestressed concrete units, seated about 100 mm onto the walls. To give some continuity and resistance to the negative moments which will occur in practice (even though, in theory, the units are 'simply' supported), it is advisable to use an rc in-situ infill within the pc floor over the wall support. This will assist in providing a robust floor slab, better equipped to resist forces due to accidental damage (see Figure 3). It is necessary to comply with the Building Regulation covering progressive collapse from accidental damage when the building is five or more storeys in height.

Where wide-span units are used to provide a fairfaced soffit, the in-situ infill shown in Figure 4 should still be provided.

The 'alternate spans loaded' condition, and the resulting bending moments and eccentricity of loading induced into the walls due to deflection of the floor units and rotation at the supports, are rarely critical. Nevertheless, the effect of eccentricity on the bearing stresses should be taken into account. The reinforcement in the in-fill tends to reduce the effect of eccentricities and distribute the uneven stresses. Many school buildings were erected in the late '50s to early '70s using high alumina cement in the precast floor units. Subsequently, all these buildings had to be investigated and, so far as the authors' experience and knowledge are concerned, none of the walls showed any distress due to eccentric loading.

Bedroom blocks
Figure 5 shows a typical basic floor plan of a bedroom block. Many buildings of this type are five to ten storeys high, and need to be checked for accidental damage under Building Regulation D17. Floors are usually in-situ continuous.

bedrooms. (It is, perhaps, regrettable that there is no similar requirement for hotel bedrooms). A half-brick wall has an average sound reduction of 42 dB and, if plastered both sides, 50 dB.

(b) Party walls - Building Regulations require 215 mm brick, or similar, between adjacent domestic units.

(c) Fire barriers - in many instances, Building Regulations require 215 mm thick brick, or similar, around staircases, lift shafts, vertical service ducts, etc, in addition to compartimenting fire breaks along the length of the building = 102.5 mm thick clay brickwork provides 2 hours fire resistance.

These functional demands dictate the need for walls which, if checked, are likely to be equally capable of fulfilling the structural function, thus eliminating the need for a structural frame.

Typical applications
Office blocks and School classroom blocks
Layouts for offices and classrooms can vary greatly, but a typical plan shape is shown in Figure 2.

The crosswalls usually need to be 215 mm thick to carry the loads. Gable and external walls are normally in 265 mm cavity brickwork. Corridor walls should be at least 102.5 mm for acoustic and fire resistance. The external and corridor walls, together with the staircase, are normally
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concrete slabs. Where the external side walls and the corridor walls are loadbearing, the floor slabs may span two ways. Some minor increase in reinforcement is all that is usually necessary to cope with the accidental damage provisions.

Crosswalls usually need to be 102.5 mm thick in order to carry the loads and to provide sound insulation. It is not uncommon to return the ends of the crosswalls, at their junctions with the external and corridor walls, to improve their stability.

Crosswall structures can, of course, be built much higher than ten storeys. However, as with all high-rise construction, the costs tend to increase faster than the increase in height.

**Low to medium-rise flats (up to six storeys)**

A typical floor plan is shown in Figure 6.

The demand for high-rise flats (which were more suited to cellular masonry construction) has waned, and there is now more interest in medium-rise blocks. These are a hybrid form of the classroom and bedroom blocks, discussed earlier, in that they tend to comprise a mixture of 215 mm and 102.5 mm crosswalls. The party walls, spaced at about 12 m centres, need to be 215 mm thick to comply with the sound requirements of the Building Regulations, and the intermediate crosswalls 102.5 mm thick to give good acoustic performance. Corridor walls and external walls are generally of masonry construction and are used to provide longitudinal stability. They may also be subject to the requirements of the Building Regulations for flanking sound transmission.

Floors are nearly always of in-situ concrete construction. Timber floors could be used in low-rise construction, if fire regulations permit. It should be remembered that the requirements for the stripping and tying of timber floors are different from and greater than those for concrete floors (see Appendix C, BS 5628, and Structural Masonry Designers' Manual). Care should be taken to ensure that the floor construction forms an efficient acoustic barrier.

**Common factors influencing design considerations of all forms of multi-storey structures**

A résumé of the more common factors which have to be considered when designing crosswall and other multi-storey structures is given below, and each item is then considered in greater detail.

1. **Stability.** A building must be stable under vertical and horizontal (wind) loads on both its longitudinal and lateral axes. Consideration must be given to the effect of openings in the walls on the stiffness of the building and the design of the shear walls.

2. **Accidental damage.** The design should take account of good engineering practice, and for buildings of 5 or more storeys comply with D17 of the Building Regulations.

3. **External walls.** Support and restraint of the outer leaf is necessary, even where the wall is non-loadbearing. This should not be confused with design against accidental damage.

4. **Concrete roof slab/wall connections.** In-situ concrete roof slabs should not usually be cast directly onto masonry walls. As the roof expands and contracts, due to thermal and other movements, the wall will tend to crack, particularly at the connection. A sliding joint should be formed between the walls and the roof slab.

5. **Choice of brick and mortar.** Whilst it is quite simple to design every wall in every storey height with a different brick and mortar, this increases the costs, planning and supervision of the contract. On the other hand, although the use of only one brick laid in one class of mortar simplifies planning and supervision enormously, it may not be the most economical solution overall. Thus, before making a choice, the cost implications should be carefully considered.

6. **Movement joints.** As with other structural
materials, movement joints must be incorporated in the structure. Whilst brickwork structures can provide a certain amount of resistance to damage due to movement, it is still necessary to install movement joints.

7. Provision for services. Early planning of service runs is necessary, so that openings in brickwork frames can be built in.

8. Vertical alignment of loadbearing walls. For simplicity, speed of construction and cost considerations, walls should remain in the same vertical plane from foundations to roof. Where, for special reasons, the occasional wall cannot be lined up, it is not difficult to accommodate such plan changes – though it does tend to increase costs.

9. Foundations. The foundations for loadbearing brick structures are generally simpler than those for structural frames. The loads are spread along the walls, founded on strip footings, so that contact pressures are low. In framed structures, loads are often concentrated at the column points, so that contact pressures are high.

10. Flexibility. Occasionally, over a period of time, there is a need to alter a structure to meet changed functional requirements. In many situations, brickwork structures are more readily adaptable to alteration than steel or concrete frames.

Stability

Figure 7 shows the main forces acting on a structure.

**Vertical stability**

It is rare for vertical instability, ie, collapse or cracking of masonry under vertical loads, to be a major problem – provided, of course, that the compressive stresses in the brickwork are kept within the allowable limits, the necessary restraints to prevent buckling are provided, and the walls are founded on adequate foundations.

**Horizontal stability (at right angles to the crosswalls)**

The wind acts on the external walls or cladding panels. These transfer the wind force to the floors and roof which, acting as horizontal plates, in turn transfer the force to the transverse walls (see Figure 8). The wind force creates racking in the transverse walls (generally termed shear walls), as shown in Figure 9, but such walls are highly resistant to racking stresses. The racking stresses are usually either eliminated by the vertical compressive load on the walls, and/or resisted by the allowable tensile stresses in the masonry. If the tensile stress should exceed the allowable limits, consideration should be given to reinforcing or post-tensioning the walls.

The stresses at the base of the wall are due to the combined effect of the vertical loading and the moment induced by the wind force, and are determined using the normal elastic stress distribution formula (see Figure 10):

\[ f = \frac{W}{A} \pm \frac{M}{Z} \]

There is usually little danger, in properly planned multi-storey masonry structures, of walls
overturning, or failing in horizontal shear, although this does depend on the designer's skill in producing a suitable layout.

Multi-storey masonry structures tend to rely for their stability on their own weight in resisting horizontal forces due to wind. They are not capable, as can be steel or concrete frames, of being considered as fully rigid frames for design purposes. In steel or concrete structures, rigidly joined frames tend to be necessary to resist lateral wind loading. It is not usually possible to develop as much rigidity at the junctions of brick walls and concrete floor slabs as there can be, for example, between in-situ concrete columns and beams. However, this is very rarely a difficult problem to overcome if sufficient forethought is given to the plan form and the structural layout.

**Longitudinal stability**

Unstiffened crosswall structures – i.e., crosswalls without stiffness at right angles to the plane of the wall – may not be stable under longitudinal loading from wind, and could collapse like a house of cards (Figure 11).

To prevent such action, longitudinal bracing is necessary. This is usually provided (see Figure 12) by either:

(a) corridor walls
(b) longitudinal external walls
(c) stiff vertical box sections formed by the walls to staircase, lifts and service ducts, or
(d) cruciform, T, Y, L-shaped block plans, or other plan forms which provide longitudinal stiffness or robustness.

**Design for wind**

In most brickwork crosswall structures, the stresses due to wind are insignificant compared to those due to dead and imposed loading, as the worked examples will show later. In a steel or concrete frame, the beams and columns are of relatively similar stiffness, rigidly connected, and are of the same material. However, with a brickwork structure, it may not act as a rigid frame because the walls are relatively sturdy and the floor slabs comparatively flimsy. For instance, in a crosswall structure with an internal corridor, the walls, being stiff, act as separate vertical cantilevers, and the corridor floors tend to...
act as pin-jointed props (see Figure 13). If both walls are of the same length, L, and thickness, t, they share the wind force equally. When they are not of equal length, they then share the wind force in proportion to their relative stiffness – if they deflect equally, as they are likely to do, because of the floors’ action in transferring the force.

The stiffness of a wall is relative to its second moment of area, \( I = \frac{tL^3}{12} \).

In the crosswall structure shown in Figure 14: wall x has an I value proportional to \( 3^3 = 27 \), wall y has an I value proportional to \( 6^3 = 216 \). Thus, wall y is eight times as stiff as wall x. Since the walls are tied together by floor slabs, they are likely to deflect equally, and wall y can be assumed to carry eight times the wind force of wall x.

The distribution of wind forces, particularly on tall slender crosswall structures, between walls of differing stiffnesses may need consideration. Some of the main points are illustrated below.

In Figure 15, the floor plan of a block of flats shows walls of differing length (and, therefore, stiffness) and of differing positions in relation to the wind. The main wind force would be resisted by walls 1, assisted by walls 2, with some help from walls 3, and little help from walls 4. An experienced designer would probably, at first, only check the effect of walls 1 and 2 in resisting wind, and then, if they were inadequate, consider the assistance of walls 3. He would be likely to ignore the minimal effect of walls 4 in resisting the wind forces. The use of walls 1 only, would necessitate a long span for the plate action of the roof or floors.

**Walls of differing section**

When external or corridor walls are bonded into crosswalls, they change the shape of a crosswall from a simple rectangular ‘plate’ section into a T, I or Z section. This can give the crosswall increased stiffness, and hence increased stability.

In Figure 16, the I and Z sections are stiffer than the T section which, in turn, is stiffer than the rectangular section.

**Openings in walls**

Intuitively, it can be seen that wall (a) in Figure 17 is stiffer than wall (b) which is stiffer than wall (c). The gable wall (d), with small widely spaced windows, may be considered to act similarly to wall (a) if the openings are relatively small. However, if the windows were deepened, the wall would approach the condition of wall (c).

Only rarely do the calculations become very complex. If they do, however, or if the designer is in any doubt as to the stiffness of the walls or structure, he should either refer to one of the many computer programmes on the market, or carry out a model test. If a computer is used, the designer should satisfy himself that the programme is suitable and well founded, and that the results of the analysis are reasonable.

**Stability of shear walls**

The stress condition for the design of shear walls
has been briefly discussed, and is based on the formula: \( f = \frac{W}{A \pm Z} \)

Brickwork design involving flexural stresses is almost invariably limited by the flexural tensile strength of the masonry. This is not surprising since the ratio of compressive strength to tensile strength is in the order of 20 to 1. Occasionally, the flexural compressive stresses can become significant, and limiting so far as the strength of certain elements are concerned. Such elements include geometrical sections such as the diaphragm wall 3 and the fin wall 4, as well as shear walls which are discussed in detail here, and fully analysed in Structural Masonry Designers' Manual 1. Flexural compressive stresses are not covered in BS 5628, but the method of analysis which follows is believed to provide a safe and reliable design, and is based on sound engineering principles.

Figure 18 shows the stress block across a wall subject to purely axial loading, and with no eccentricity of that load. The maximum compressive stress allowable in the wall section is limited by the masonry’s tendency to buckle, hence the inclusion of the capacity reduction factor, \( \beta \).

Figure 19 shows the stress block across a wall subject to purely flexural loading conditions.

BS 5628: Part 1 does not differentiate between axial compression and flexural compression. However, it is generally accepted that allowable flexural compressive stresses may be higher than allowable axial compressive stresses. The flexural tensile stresses will, as stated earlier, normally be the limiting factor. However, if the axial compressive stresses already in the wall are added to the flexural compressive stresses, this may produce a more critical design condition. Consideration must be given to the need for limiting the flexural compressive stresses, due to the possibility of the section buckling under the application of such stress (see Figure 20).

The authors consider that the following design method provides a safe and practical solution:

(a) In the first instance, the wall should be checked for maximum axial loading only, using the design principles given in BS 5628, clause 32. The capacity reduction factor, \( \beta \), applicable to this stage of the design, should be derived from the maximum slenderness ratio. The maximum allowable stress under this loading condition is:

\[ \frac{\beta f_k}{\gamma_m} \]

(b) The additional compressive stress resulting from the bending due to the lateral loading is then considered, and the maximum allowable combined compressive stress is

\[ 1.1 f_k \times \frac{\beta}{\gamma_m} \]
which a 10% increase has been applied to the flexural aspect of the stress, in a similar manner to Appendix B of BS 5628. The capacity reduction factor, $\beta$, should be derived from the slenderness ratio, which incorporates the effective thickness appropriate to the direction of the buckling tendency (ie, perpendicular to the direction of application of the bending) as shown in Figure 20.

The stress blocks for the two stages of the design are shown in Figure 21. The application of this proposed design will be demonstrated in the worked examples which follow.

**Accidental damage**

Design for accidental damage is a subject in itself, and to provide a thorough commentary on all aspects of it would require a document as large again. Readers are again referred to Structural Masonry Designers' Manual, in which a whole chapter has been devoted to this important subject, and to BS 5628, clause 37.

The design examples which follow will also briefly consider the implications of accidental damage, but it is worth repeating here the general recommendations of BS 5628, which may be interpreted as follows:

1. The designer responsible for the overall stability of the structure should ensure that the design, details, fixing, etc, of elements or parts of the structure are compatible, whether or not the design and details were made by him. All too often, the design of a building comprises a series of element designs, carried out by the respective suppliers of precast floors, timber trussed rafter roofs, steel floor beams, etc, and no one is appointed to be responsible for the overall stability. A situation which has, upon investigation by the authors, been responsible for numerous disastrous consequences in the past, and one which should not be allowed to prevail in the future.

2. The designer should consider the plan layout of the structure, returns at the ends of walls, interaction between intersecting walls, slabs, trusses, etc, to ensure a stable and robust design. The collapse of any part of a structure should not be out of proportion to the cause of the collapse, as was the case in the Ronan Point disaster of 1968. Progressive collapse is far less likely to occur in properly designed and detailed brickwork structures than the untied industrialised precast walling systems of that era.

3. The designer should check that lateral forces acting on the whole structure are resisted by the walls in the planes parallel to those forces, or are transferred by them by plate action of the floors, roof, etc, or that the forces are resisted by bracing or other means.

The structure must have adequate residual stability not to collapse completely, and the Code further advises that the designer should satisfy himself that ‘... further collapse of any significant proportion or the structure is unlikely to occur’. The structure is not necessarily required to be serviceable after the event, but collapse should be limited to provide a means of escape for the occupants and adequate stability to facilitate its demolition or rehabilitation.

**External walls**

External walls can be solid, cavity, diaphragm, fin, or have piers. It is quite common for the outer leaf of a cavity wall, or the face of a solid wall, to be in a different type of unit from the inner leaf or face. In cavity wall construction, a very frequent example is the use of a clay facing brick externally and an insulating block internally.
Note that, in the case of a solid wall with different bricks on the outer and inner faces, the bricks should have compatible movement characteristics.

Cavity walls are more popular than solid walls because they are more resistant to rain penetration, and have better thermal insulation properties. However, care and attention must be given to the choice and fixing of the wall ties. Often, the outer leaf only helps to stiffen the inner loadbearing leaf – but this action is only possible with sufficient, good and durable ties. BS 5628 advises that the external leaf of a cavity wall should be supported at least every third storey, to reduce the effect of loosening of the wall ties owing to the differential movement of the external and internal leaves. The Code allows an exception to this rule for buildings not more than 4 storeys and 12 m in height, where the outer leaf may be uninterrupted for its full height.

Whilst, to some extent, both leaves carry the wind load, in addition to carrying their own weight, the inner leaf usually supports most of the floor load. Where the outer leaf carries its self-weight only, the choice of facing brick is not usually restricted by strength requirements.

For multi-storey structures with large areas of floor spanning onto external walls, the loads on the walls may be very high. Their strength and thermal requirements may appear to conflict. There are at least two possible solutions to this problem:
(a) carry the floor loads on a dense inner leaf, using insulation in the cavity;
(b) carry the floor loads on the outer leaf, its specification unchanged if appropriate, so making the insulation independent of the structure.

When using a non-structural block for the inner leaf, consideration should be given to providing a flexible joint to prevent loads being transferred into it.

When the floor slab bears on a half-brick outer leaf, it is preferable to carry it across to the outside face. If it is considered necessary to mask the slab, this may be achieved with brick slips. Details of types, fixings, etc. can be found in modern text books on building construction. A typical detail is shown in Figure 22. If the floor slab bears on a one brick or thicker wall, the floor slab can be masked by a course of bricks, see Figure 23.

Engineers tend to prefer bricks rather than slips, and to anchor them (and thus restrain the outer leaf) to the slab by anchor ties or steel angles, as shown in Figure 24.

With current thermal requirements, insulation will usually be within the cavity. Care should be taken, when carrying the floor on the outer leaf, to minimise the effects of cold bridging at the floor/wall junctions.

The external walls in Figure 12(b) may be subject to high lateral loads combined with only minimal vertical loads. Such brick walls do not have a high resistance to bending perpendicular to their plane. The wall panels in the top storey are most at risk, because they are likely to be subject to the greatest wind pressure whilst the only compensating precompression is the vertical loading from the roof and the wall’s own weight. If a lightweight timber roof is used, there could be wind uplift forces, which would have to be counteracted by strapping the roof down to the walls. There would then be no vertical precompression in the top storey walls.

Generally, this is not a significant problem with loadbearing brickwork – but it can be, if the brickwork is non-loadbearing and is used merely (and wastefully) as a cladding to a steel or concrete-framed structure. However, where loadbearing panels do lack sufficient precompression, the problem can be overcome by

Loadbearing brickwork crosswall construction
post-tensioning the wall (Figure 25). The relatively small diameter rods can be anchored into the floor slab below, at regular, designed centres, and extend up the wall panel through the cavity, where they should be provided with some form of corrosion protection, preferably a proprietary tape. The rods are anchored again at the head of the wall panel, possibly onto a concrete ring beam, roof slab or padstone, through a steel bearing plate. By means of nuts on their threaded ends, the rods can be tensioned to a designed value, using a simple torque wrench, with due allowances being made for the various forms of losses. In this way, the previously inadequate precompression can be increased to assist the wall’s stability.

Concrete roof slab/loadbearing wall connection
Whilst it is good practice, and structurally beneficial, to cast floor slabs onto the walls, it is inadvisable to cast the roof slab directly on the top of the upper storey wall. The roof slab will tend to expand and contract with temperature variations, and if it is restrained by the slab/wall connection, either it or the wall will crack.

In order to reduce this effect, the roof slab should be separated from the supporting wall. An effective separation joint can be achieved by inserting two layers of dpc (see Figure 26) or a proprietary jointing material. It is essential that the joint is flat, otherwise a slip plane will not be formed.

Choice of brick and mortar strengths
Usually, the bottom storey masonry will be the most highly stressed. The stress diminishes at each storey height, and the top storey is usually the most lightly stressed.

Inevitably, within any one storey height, some walls will be more heavily stressed than others. For example in, say, a six-storey hostel block, the crosswalls may be 102.5 mm thick and the walls surrounding the staircase may, for fire protection purposes, be 215 mm thick whilst only carrying the same load as the crosswalls. Thus, it follows that every storey height could be of a different strength of brickwork, and that, within any one storey height, variations in brickwork strength could be employed. However, any savings in material costs due to the widespread variation would be swallowed up by the extra costs of organising, sorting, stacking, supervising, etc.

It is generally advisable to use a maximum of only three mortar strengths: 1:1:3 below dpc level and in extremely highly stressed work; 1:1:6 (or 1:1:4) for external and highly stressed work; 1:2:9 for internal work (ie, BS 5628 mortar designations (i), (iii) and (iv) respectively).

It is difficult for administrative or supervisory staff to check the strength of the bricks and the mortar mix by sight. Reducing the cement content of the mortar only produces a minimal saving in the cost per m³ of the masonry. Every effort should be made to keep the walls of a constant thickness throughout their height. It should be kept in mind that a slender, highly stressed wall is usually cheaper than a thick wall carrying a low stress. Brickwork strengths should generally be uniform throughout any one storey, and changes in strength should be limited to approximately every three storeys.

Note that a top storey wall, due to its small pre-load, may have excessive flexural tensile stress resulting from wind forces, and may require specific brick and mortar strengths to cope with this.

Movement joints
On long crosswall structures, it is essential to insert movement joints to counter the effects of thermal and moisture movements. They are also advisable on structures liable to undergo excessive differential settlement and mining subsidence. Movement joints should also be used to break up L and T plan shapes, and other similar building
configurations, when they are sensitive to movement. A typical method of achieving this, in crosswall structures, is shown in Figure 27.

Services, finishes, etc, which have to cross the movement gap should be provided with flexible connections, as in concrete or steel-framed structures. The spacing of movement joints must relate to their function, and there are no rigid rules applicable to the determination of spacing. For example, to provide for moisture and thermal movement of the masonry, 12 m spacing of control joints is usually adequate for clay brickwork, whereas much closer spacing, say 5 m or 6 m, is necessary for calcium silicate brickwork. Further advice is given in CP 121.

Settlement and mining movement joint centres can only be assessed from a consideration of the relevant sub-soil and mining information.

**Provision for services**

Inevitably, pipes for hot and cold water supply, conduits for electrical cables, air-conditioning ducts, etc, have to pass through loadbearing brick walls. The openings or holes for these services must always be pre-planned. Services engineers are accustomed to indiscriminate breaking out of large holes and cutting chases in relatively thick walls of traditional brickwork construction, when upgrading or changing the services in existing buildings. They do not always appreciate that ad hoc alterations cannot be permitted in modern, slender, highly stressed walls. Holes and chases should not be cut without the prior approval of the structural designer.

Pre-formed openings can easily be arranged by leaving out bricks when building the wall. If the openings are large, or could cause over-stressing or undesirable stress concentration in the surrounding masonry, reinforcement can be laid in the bed joints above the openings – and around, if necessary – to distribute the stress. Detailed drawings of service holes and chases should be given to the contractor before the commencement of building operations. A typical builders-work drawing is shown in Figure 28.

Chases should be sawn out to the depth agreed by the structural designer, and not be hacked out by hammer and chisel except in special cases. Horizontal or diagonal chases are rarely permissible in highly-stressed zones, since they tend to reduce the effective cross-sectional area and increase the buckling tendency of the wall. Nor are vertical chases usually permissible in half-brick thick walls (102.5 mm) without careful design checks, since they can induce vertical splitting in the masonry.

Holes for vertical service runs through floor slabs form a very useful site aid in setting out and checking the vertical alignment of walls. Vertical ducts can easily be formed by making minor adjustments to the wall layouts (see Figure 29).

**Vertical alignment of loadbearing walls**

Whilst engineers and architects have long accepted the need for column grid layouts, and are well aware of the need to line-up columns from foundations to roof (ie, column positions should, where possible, remain constant), they do not, at first, readily accept the same discipline.
in brickwork structures – no doubt because they have been used to placing non-loadbearing walls or partitions anywhere.

Non-loadbearing partitions can still be placed practically anywhere in a loadbearing brickwork frame. Nevertheless, as with steel or concrete columns, it is desirable that the loadbearing walls are lined up. They can, of course, be moved out of line – but this may mean expensive and complex beam and beam-support layouts. This factor, more than any other, has tended to militate against the use of loadbearing brickwork, especially in situations where the ground floor layout differs from the upper floors. For example, in a hotel bedroom block, the ground floor may require large open spaces for restaurant, reception areas, etc. The conflicting needs of the ground floor and the upper floors can easily be reconciled by podium construction (see later).

The authors’ experience has shown that designers quickly adapt to the need for planning discipline, and welcome the benefits of repetition of floor layouts, windows, doors and other furniture, service runs, finishes, etc, with the accompanying savings in cost and erection time, and the simplicity of construction.

Brickwork structures can accommodate a wide range of functional requirements. It is simply a question of choosing the form best suited to the function.

Foundations
The narrowest strip footing that can be conveniently dug by an excavator usually results in a foundation area such that the soil contact pressures are low. For example, a nine-storey hostel block with 102.5 mm crosswalls, founded on a 600 mm wide concrete strip footing, would have a contact pressure of only about 325 kN/m². When the ground bearing capacity is so low that piling is necessary, the wall itself can be treated as the compression flange of a composite reinforced concrete/masonry ground beam, with attendant savings in foundation costs (see Figure 30). It should be noted, however, that the use of a wall as a composite beam may partially limit its adaptability should it become necessary to change the structure at a later date.

Because brick walls are pliant, compared to structural steel or rc frames, they are particularly economical on sites subject to mining or other subsidence. Reinforcing the lower and upper bed joints at each storey height results in a wall that is highly resistant to differential settlement, although care must be taken to ensure full and adequate cover to the reinforcement.

Flexibility
Many designers think that brickwork structures are inflexible – that it is difficult to alter them, once they are built. This is not so. For example,
one of the authors’ most interesting change-of-use projects was the successful conversion of a Victorian ice-cream factory into an old people’s home.

The spate of conversion, alteration and rehabilitation of brickwork structures in the ’70s gave masonry designers the opportunity to prove that it is often easier to alter a brickwork structure than a steel or concrete structure. It is often easier to demolish a brick wall than a steel or concrete column. And it is far simpler to form an opening in a brick wall than in a reinforced concrete wall. Generally, it is cheaper to bond in, thicken, brace, or otherwise strengthen a brick wall than a steel column. It is often easier and quicker to repair an overloaded brick wall or arch than the equivalent in steel or concrete.

Although alterations to modern, highly-stressed, loadbearing brickwork structures require careful attention, it is only on rare occasions, when wholesale alterations are required for a radical change of use, that brickwork structures become inflexible.

**Elevational treatment of crosswall structures**

Long side walls pierced by hole-in-the-wall windows can be visually dull. There are many ways of overcoming this – for example by using decorative brickwork and/or modelling the elevation (see Figure 31).

**Speed of erection**

The speed of construction of crosswall building is very impressive, particularly if the plan form and size of the structure allow it to be constructed in quarters, using the sequential method whereby the trades can follow each other around the building from one quarter to the next as they complete their section of the work (see Figure 32).

From the stages indicated in the diagram, the bricklayers on completion of Bay 4 would move up to the next floor and start work in Bay 1. The other trades, ie, shutterers, steel fixers and concretors, would all move on one bay – the construction continuing to spiral up the building, keeping all trades constantly employed.

**Podium construction**

A common objection to the use of crosswall construction is that the ground floor planning requirements often demand more open space than crosswalls permit. Typical examples are reception areas and restaurants in hotels, car parking for flats, recreation areas and shops in student hostels. But the floors above, with regular wall layouts, are ideal for crosswall construction.

Frequently, there is no need to frame the whole structure, merely because of the ground floor planning requirements. A different structural form can be used for the ground storey, and a common solution to the problem is to form a podium with steel, concrete or brickwork columns supporting a concrete deck, as shown in Figure 33. Depending on the load from the crosswalls, the deck can be of plate or waffle slab construction, diagrid or T-beam. The deflection of the deck under the crosswalls should be assessed, even though clay brickwork often has an inherent flexibility that enables it to adjust to the deflection of a concrete beam.

**Partitions**

Where non-loadbearing brick partitions are built parallel to long span floor slabs, particularly where prestressed concrete floors are employed, the deflection of the floor may be of such magnitude as to cause cracking in the masonry. This is generally caused by the brickwork attempting to arch over the deflected floor – which problem can be minimised by the introduction of a dpc membrane beneath the partition, and the addition of bed joint reinforcement in lower courses, as indicated in Figure 34.

**References**


Above WOMEN'S HALL OF RESIDENCE, BANGOR UNIVERSITY

By the time this design was started (1960) and following the experience at St John Rigby (page 2) and subsequent loadbearing brickwork schools, the designers were sufficiently experienced in structural brickwork design to be confident that 9in and 6in (Calculon) walls were unnecessarily thick, and that 4\text{\textfrac{1}{2}}in thick walls would be adequate.

However, no test data on such slender crosswalls was available. Calculations and design assumptions were checked and re-checked, and compression tests carried out by Professor A. W. Hendry on storey-height panels at Liverpool University confirmed expectations. Later, after completion of the building, a full-scale model of the structure was built and tested by Professor Hendry at Edinburgh University.

A major problem at the start of the contract was in achieving the standard of brickwork specified. Neither the site agent nor the clerk of works could convince the operatives of the more exacting standards of brickwork required. Finally W. G. Curtin tried with slides of research work calculations, specification clauses, and a cardboard model which showed how the structure would work. The message got through. The standard of work shot up, and the turnover of labour ceased. So good was the standard of bricklaying, that the client agreed to the erection of a panel of brickwork carrying the initials of the bricklayers.

Architects Colwyn Foulkes & Partners
Structural engineers W. G. Curtin & Partners
**Above** LINNET LANE SHELTERED HOUSING, LIVERPOOL

A single block of 32 flats, visually divided by a recess on the front elevation to give the appearance of two domestically-scaled units. Half-brick and one-brick thick crosswalls support precast concrete floors, with local insitu areas giving support to cantilevered balconies. Reinforced concrete footings on vibro-compacted sub-strata.

*Architects* Building Design Group  
*Structural engineers* W. G. Curtin & Partners

**Right** CHRISTOPHER GRANGE BLIND INSTITUTE, WEST DERBY, LIVERPOOL

An early use of podium construction. Ground floor is an open plan rc framed area providing reception, meeting, occupational therapy areas, chapel, etc. There was no need to carry the frame through to the bedroom and flat block over – as is usually done. Loadbearing brick crosswalls built off the rc podium provided a much more economical solution.

*Architects* Roy Croft & Partners  
*Structural engineers* W. G. Curtin & Partners

**Left** ST PETER'S COURT SHELTERED HOUSING, ROCK FERRY


*Architects* Merseyside Improved Housing Architects  
*Structural engineers* W. G. Curtin & Partners
Requirements were for a development providing premises for the Midland and National Westminster banks and three shop units, premises for the Joint Service Units, and residential accommodation for students in such a form as to be suitable for use by either undergraduates, postgraduates, married students and staff.

The site lies between buildings diverse in form and character, and thus posed difficult problems in the creation of a good visual relationship between the new buildings and their existing neighbours. To deal with the problem of scale, the two lower storeys were visually combined into one thus helping to achieve a transition from the large scale of Mount Pleasant on one side and the smaller one of the precinct on the other.

The structural form is podium construction with a concrete frame up to first floor level and loadbearing brickwork above. Allowance was made for differential movement between the ground floor framed structure and the brick structure by dividing the residential block into units and providing movement joints across the building.

Architects Manning Clamp & Partners
Structural engineers W. G. Curtin & Partners
Left and above SUB-DIVISION POLICE HEADQUARTERS, ELLESMERE PORT
Loadbearing brick columns (some backed by internal crosswalls) with rc edge beams spanning between. Half-brick and one-brick thick internal crosswalls. Accommodation incorporates large open areas. A similar sized steel-framed structure was started nearby at the same time. By the time the steel frame had been encased in concrete, the police were moving into their loadbearing brick structure.
Architects Paterson Macaulay & Owens
Structural engineers W. G. Curtin & Partners

Below and left below BAPTIST MEN’S MOVEMENT SHELTERED ACCOMMODATION, PRINCES AVENUE, LIVERPOOL
A five- and four-storey block with a mansard roof, built in a conservation area and designed to blend with the adjacent properties. Insitu concrete floors carried on loadbearing brick crosswalls. Cantilevered window details. Provisions for vertical expansion incorporated in external walls. Reinforced footings on vibro-compacted fill to old church basement.
Architects David Parry, Quiggin & Gee Associates
Structural engineers W. G. Curtin & Partners
Left and above BLAIR COURT, BIRKENHEAD

A six-storey block of flats at the edge of Birkenhead Park for Merseyside Improved Housing Association. Built on a sloping site, the front entrance is at ground level, with a basement opening out to a lower level at the rear of the building. Loadbearing half-brick and one-brick thick crosswalls support insitu rc floors. Common room in the basement area incorporates rc frames. Mansard roof at top level to conform to adjacent building heights.

Architects Paterson Macaulay & Owens
Structural engineers W. G. Curtin & Partners

Above and right GAMBIER TERRACE, LIVERPOOL

Part of a fine Regency terrace overlooking the Anglican cathedral. Unfortunately, the building was crumbling and practically beyond repair. It was decided to preserve the facade, demolish the rest of the building, erect a new structure and pin the restored facade to it. Scheme comprises a four- and six-storey block of sheltered housing, with semi-insitu floors spanning onto loadbearing brick crosswalls. Original facade was temporarily propped – final restraint being provided by the six-storey element.

Architects David Parry, Quiggin & Gee Associates
Structural engineers W. G. Curtin & Partners
HOSTEL BUILDING 9-STOREYS HIGH

Building geometry (see Figures 34 & 35)

Overall height = 24.30 m
Overall length = 32.00 m
Overall width = 14.00 m
Floor to floor height = 2.70 m
Span of rc floors = 4.00 m (150 mm thick in-situ concrete).

Assume: masonry density = 19.00 kN/m³
Concrete density = 24.00 kN/m³

Roof and floor slabs are 150 mm thick in-situ reinforced concrete. External facing bricks selected have a water absorption of 7%, and a compressive strength of 35 N/mm². Designation (iii) mortar (1:1:6) will be used externally throughout. Extensive quality control and testing of materials will be carried out, and supervision of the reputable contractor will be maintained at all times.

Characteristic loads

**Roof:** dead load, \( G_k \), 150 mm slab = 3.60
screed to falls, say = 1.00
imposed load, \( Q_k \), (no direct access) = 4.60 kN/m²

**Floors:** dead load, \( G_k \), 150 mm slab = 3.60
partitions = 0.90
services, etc = 0.20
finishes, etc = 1.30
imposed load, \( Q_k \), (bedrooms) = 6.00 kN/m²
2.00 kN/m²

Wind loading: is assumed to have been calculated on the basis of CP 3: Chapter V: Part 2: 1972, to give a maximum characteristic wind pressure on the walls of + 0.90 kN/m², and a maximum gross characteristic wind uplift on the flat roof of + 1.25 kN/m².

Design of typical internal crosswall

**Loading**

<table>
<thead>
<tr>
<th>Characteristic dead loads floors &amp; roof</th>
<th>kN/m</th>
<th>Characteristic imposed load (4 x 0.75)</th>
<th>kN/m</th>
<th>Imposed load less reductions</th>
<th>kN/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>roof (4 x 4.6)</td>
<td>18.4</td>
<td>(4 x 0.75)</td>
<td>3.0</td>
<td>0%</td>
<td>3.0</td>
</tr>
<tr>
<td>8th 18.4 + (4 x 6.0)</td>
<td>42.4</td>
<td>3 + (4 x 2.0)</td>
<td>11.0</td>
<td>10%</td>
<td>9.90</td>
</tr>
<tr>
<td>7th 42.4 + 24.0</td>
<td>66.4</td>
<td>11 + 8</td>
<td>19.0</td>
<td>20%</td>
<td>15.20</td>
</tr>
<tr>
<td>6th 66.4 + 24.0</td>
<td>90.4</td>
<td>19 + 8</td>
<td>27.0</td>
<td>30%</td>
<td>18.90</td>
</tr>
<tr>
<td>5th 90.4 + 24.0</td>
<td>114.4</td>
<td>27 + 8</td>
<td>35.0</td>
<td>40%</td>
<td>21.00</td>
</tr>
<tr>
<td>4th 114.4 + 24.0</td>
<td>138.4</td>
<td>35 + 8</td>
<td>43.0</td>
<td>40%</td>
<td>25.80</td>
</tr>
<tr>
<td>3rd 138.4 + 24.0</td>
<td>162.4</td>
<td>43 + 8</td>
<td>51.0</td>
<td>40%</td>
<td>30.60</td>
</tr>
<tr>
<td>2nd 162.4 + 24.0</td>
<td>186.4</td>
<td>51 + 8</td>
<td>59.0</td>
<td>40%</td>
<td>35.40</td>
</tr>
<tr>
<td>1st 186.4 + 24.0</td>
<td>210.4</td>
<td>59 + 8</td>
<td>67.0</td>
<td>40%</td>
<td>40.20</td>
</tr>
</tbody>
</table>

From previous experience it is expected that a 102.5 mm thick wall will be adequate for the full height Loadbearing brickwork crosswall construction
of the internal crosswalls. Less experienced designers are referred to Structural Masonry Designers’ Manual\textsuperscript{1} for guidance on obtaining trial wall sections, and for tables of design loads for solid walls. The masonry strengths required will be calculated at the four levels marked with an asterisk in Figure 35 (ie, lowest, fourth and seventh storeys).

The design vertical strength of the internal crosswalls will be assessed initially and, for this, the loading combination of dead plus imposed will be considered:

hence, \( y_r = 0.9 \, G_k \) or \( 1.4 \, G_k \) and \( \rho = 1.6 \, Q_k \) (BS 5628, clause 22).

**Design loads (including own weight of masonry)**

At level A:
- dead load, floors & roof = 210.4 \times 1.4 = 294.56
- imposed load = 40.2 \times 1.6 = 64.32
- dead load masonry = 0.1025 \times 19 \times 24.3 \times 1.4 = 66.25
- total \( n_w = 425.13 \, \text{kN/m} \)

At level B:
- dead load, floors & roof = 138.4 \times 1.4 = 193.76
- imposed load = 25.8 \times 1.6 = 41.28
- dead load masonry = 0.1025 \times 19 \times 16.2 \times 1.4 = 44.17
- total \( n_w = 279.21 \, \text{kN/m} \)

At level C:
- dead load, floors & roof = 66.4 \times 1.4 = 92.96
- imposed load = 15.2 \times 1.6 = 24.32
- dead load masonry = 0.1025 \times 19 \times 8.1 \times 1.4 = 22.08
- total \( n_w = 139.36 \, \text{kN/m} \)

**Capacity reduction factor**

The in-situ concrete floor slab can be assumed to provide enhanced resistance to lateral movement. Hence, the effective height of the wall may be taken as 0.75 times the clear height (BS 5628, clause 28.3.1.1).

Therefore, effective height = \((2.70 - 0.15) \times 0.75 = 1.91 \, \text{m}\)

and

slenderness ratio = \( \frac{h_{ef}}{t_{ef}} = \frac{1.91}{0.1025} = 18.6 \)
The proportion of dead load to imposed load in this example will ensure that the resultant eccentricity of the loading system will be within 0.05t, and therefore will not influence the capacity reduction factor. This can be verified by a simple calculation of the theoretical position of the resultant load, based on the assumptions given in BS 5628, clause 31.

The dead and imposed loads above the slab level under consideration may be taken as axial, and the alternate spans loaded situation will be analysed. Thus, the loading arrangement shown in Figure 36 is appropriate to the maximum possible eccentricity of load.

![Calculation of eccentricity of load](image)

Check the condition at level A.

\[ R_1 = (1.4 \times 6 \times 2) + (1.6 \times 2 \times 2) = 16.8 + 6.4 = 23.2 \text{ kN/m} \]
\[ R_2 = (0.9 \times 6 \times 2) = 10.80 \text{ kN/m.} \]
Minimum load in wall above 1st floor slab
\[ = 0.9 (186.4 + 0.1025 \times 19 \times 21.6) = 205.62 \text{ kN/m.} \]
Resultant position from left hand face of wall
\[ = \frac{205.62 \times 0.051 + 23.2 \times 0.017 + 10.8 \times 0.085}{205.62 + 23.2 + 10.8} = 0.0493 \text{ m.} \]
Therefore, eccentricity from \( \theta \) at top of wall
\[ e_x = 0.051 - 0.0493 = 0.0017 \text{ m.} \]
Check the condition at level C.

\[ R_1 = (1.4 \times 6.0 \times 2) + (1.6 \times 2.0 \times 2) = 16.8 + 6.4 = 23.2 \text{ kN/m} \]
\[ R_2 = (0.9 \times 6 \times 2) = 10.80 \text{ kN/m.} \]
Minimum load in wall above 7th floor slab
\[ = 0.9 (66.4 + 0.1025 \times 19 \times 5.4) = 69.22 \text{ kN/m.} \]
Resultant position from left hand face of wall
\[ = \frac{69.22 \times 0.051 + 23.2 \times 0.017 + 10.8 \times 0.085}{69.22 + 23.2 + 10.8} = 0.0469 \text{ m.} \]
Therefore, eccentricity from \( \theta \) at top of wall
\[ e_x = 0.051 - 0.0469 = 0.0041 \text{ m.} \]
\[ e_x = 0.0041t, \text{ ie, less than } 0.05t. \]
Then, at all levels up to level C:
with slenderness ratio \( = 18.6 \)
and \( e_x = 0 \) to 0.05t.
from BS 5628, table 7, \( \beta = 0.749 \)
(above level C, assume an eccentricity of 0.1t.)

**Partial safety factor for material strength**

From BS 5628, clause 27, both manufacturing and construction control can be classed as 'special' for load-bearing brickwork crosswall construction.
the conditions described at the beginning of this design example. Thus, from BS 5628, Table 4, $\gamma_m = 2.5$.

This value must be assessed by the designer, based upon the conditions prevailing, for each individual project.

**Masonry strengths required**

From BS 5628, clause 32.2.1:

\[
\frac{\beta t f_k}{\gamma_m} \geq n_w
\]

Therefore, characteristic strength of masonry required, $f_k$,

\[
= \frac{n_w \gamma_m}{\beta t}
\]

For narrow brick walls, BS 5628 permits a stress increase factor of 1.15 (see BS 5628, clause 23.1.2). Therefore, characteristic strength of masonry required, $f_k$,

\[
= \frac{n_w \gamma_m}{1.15 \beta t}
\]

At level A:

characteristic strength of masonry required, $f_k$,

\[
= \frac{425.13 \times 2.5 \times 10^3}{1204} \approx 7.91 \text{ N/mm}^2.
\]

Use bricks with a compressive strength of 27.5 N/mm², set in a designation (ii) mortar.

At level B:

characteristic strength of masonry required, $f_k$,

\[
= \frac{279.21 \times 2.5 \times 10^3}{791} \approx 39.36 \text{ N/mm}^2.
\]

Use bricks with a compressive strength of 20 N/mm², set in a designation (ii) mortar.

At level C:

characteristic strength of masonry required, $f_k$,

\[
= \frac{139.36 \times 2.5 \times 10^3}{393.6} \approx 3.95 \text{ N/mm}^2.
\]

Use bricks with a compressive strength of 10 N/mm², set in a designation (iii) mortar.

It may be noted that the majority of bricks classified as ‘commons’ have a compressive strength of at least 20 N/mm², although some calcium silicate bricks can be of lower strength. In practice, therefore, it may be considered that the required compressive strength of 10 N/mm² is an unreasonably low specification.

Check characteristic wall strength above level C:

with slenderness ratio $= 18.6$ and $\varepsilon_s = 0.1t$.

from BS 5628, Table 7, $\beta = 0.682$.

Design loads at level D:

dead load, floors & roof $= 42.4 \times 1.4 = 59.36$

imposed load $= 11.0 \times 1.6 = 17.6$

dead load masonry $= 0.1025 \times 19 \times 5.4 \times 1.4 = 14.72$

\[
= 91.68
\]

Characteristic strength of masonry required, $f_k$,

\[
= \frac{91.68 \times 2.5 \times 10^3}{1150} = 2.85 \text{ N/mm}^2.
\]

Therefore, brick and mortar strength specified for level C is adequate. The masonry specification, reducing in the upper storeys, is shown in Figure 37.
the conditions described at the beginning of this design example. Thus, from BS 5628, Table 4, $\gamma_m = 2.5$.

This value must be assessed by the designer, based upon the conditions prevailing, for each individual project.

**Masonry strengths required**

From BS 5628, clause 32.2.1:

$$\frac{\beta f_k}{\gamma_m} \geq n_w$$

Therefore, characteristic strength of masonry required, $f_k$, is

$$f_k = \frac{n_w \gamma_m}{\beta t}$$

For narrow brick walls, BS 5628 permits a stress increase factor of 1.15 (see BS 5628, clause 23.1.2). Therefore, characteristic strength of masonry required, $f_k$, is

$$f_k = \frac{n_w \gamma_m}{1.15 \beta t}$$

At level A:

characteristic strength of masonry required, $f_k$:

$$2425.13 \times 2.5 \times 10^3$$

$$= \frac{1.15 \times 0.749 \times 102.5 \times 1000}{12.04 \text{ N/mm}^2}$$

Use bricks with a compressive strength of 27.5 N/mm², set in a designation (ii) mortar.

At level B:

characteristic strength of masonry required, $f_k$:

$$279.21 \times 2.5 \times 10^3$$

$$= \frac{1.15 \times 0.749 \times 102.5 \times 1000}{7.91 \text{ N/mm}^2}$$

Use bricks with a compressive strength of 27.5 N/mm², set in a designation (ii) mortar.

At level C:

characteristic strength of masonry required, $f_k$:

$$139.36 \times 2.5 \times 10^3$$

$$= \frac{1.15 \times 0.749 \times 102.5 \times 1000}{3.95 \text{ N/mm}^2}$$

Use bricks with a compressive strength of 10 N/mm², set in a designation (iii) mortar.

It may be noted that the majority of bricks classified as ‘commons’ have a compressive strength of at least 20 N/mm², although some calcium silicate bricks can be of lower strength. In practice, therefore, it may be considered that the required compressive strength of 10 N/mm² is an unreasonably low specification.

Check characteristic wall strength above level C:

with slenderness ratio $= 18.6$

and $\varepsilon_e = 0.1t$.

from BS 5628, Table 7, $\beta = 0.682$.

Design loads at level D:

dead load, floors & roof $= 42.4 \times 1.4 = 59.36$

imposed load $= 11.0 \times 1.6 = 17.6$

dead load masonry $= 0.1025 \times 19 \times 5.4 \times 1.4 = 14.72$

$$91.68$$

Characteristic strength of masonry required, $f_k$:

$$91.68 \times 2.5 \times 10^3$$

$$= \frac{1.15 \times 0.682 \times 102.5 \times 1000}{2.85 \text{ N/mm}^2}$$

Therefore, brick and mortar strength specified for level C is adequate. The masonry specification, reducing in the upper storeys, is shown in Figure 37.
Design of external cavity wall for wind
The critical design case will occur on the top storey at the gable, where the minimum compression on the wall is further reduced due to wind uplift pressures on the roof slab. Walls subject to high lateral loading, and low compressive load, are more likely to fail due to flexural tensile stresses, rather than axial compressive stress or buckling.

Consider roof uplift
minimum roof dead load = 0.9 × 4.6 = 4.14 kN/m²
maximum wind uplift on roof = 1.4 × 1.25 = 1.75 kN/m²
nett roof dead load (after uplift) = 4.14 × 1.75 = 2.39 kN/m²
nett roof dead load on wall = 2.39 × 2 = 4.78 kN/m

Design method
BS 5628 acknowledges that there is not a precise design method for such walls, but suggests two approximate methods:
(a) designing as a wall panel supported on a number of sides;
or,
(b) designing as an arch spanning between supports.

Regarding the second option, for the wall under consideration there is insufficient dead load available to resist an arch thrust in the vertical plane. In any case, the authors consider that the option of designing as an arch can be difficult to justify, and should not generally be used.

Taking the first option, BS 5628, clause 36.4.3, gives the flexural strength of vertically loaded panels as:

\[
\frac{f_{kx}}{\gamma_m} Z, \quad \text{where}
\]

- \( f_{kx} \) is the characteristic flexural strength,
- \( \gamma_m \) is the partial safety factor for materials,
- \( Z \) is the section modulus.

No account is taken of the considerable assistance to this resistance moment that is provided by the vertical compressive loads.

The authors consider that the following design method, in which the applied bending moments are assessed from basic principles and the compressive loads in the brickwork are exploited, provides a safe and practical design based on sound and reliable engineering principles.

Stability moment of resistance
Clause 36.5.3 of BS 5628 gives the design moment of resistance for free-standing walls as:

\[
 \left( \frac{f_{kx}}{\gamma_m} + g_d \right) \times Z,
\]

in which the assistance provided by the axial compressive stress, \( g_d \), is exploited. However, this formula is based on elastic analysis, and is limited by the flexural tensile resistance which may, in fact, be zero at the base of the wall if a felt dpc is present.

The stability moment of resistance concept exploits the gravitational mass of the brickwork, plus any nett roof loads, to generate a 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 38.
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 brickwork, \( w_s \), is stressed to the ultimate to produce the maximum lever arm for the axial load to generate the maximum stability moment of resistance \( M_{Res} \). The ultimate stress applied to this minimum width of brickwork is termed the ‘allowable flexural compressive stress’, \( P_{ubc} \).

\[
P_{ubc} = \frac{1.1 f_k \beta}{\gamma_m},
\]

in which

- \( 1.1 \) is the stress increase factor to take account of the flexural aspect of \( f_k \),
- \( f_k \) is the characteristic compressive stress,
- \( \gamma_m \) is the partial safety factor for materials,
- \( \beta \) is the capacity reduction factor which, owing to the restraint available at floor level, is taken as 1.0.

For this design example, the axial load differs in each leaf of the cavity wall, and the total stability moment of resistance will be equal to the sum of the stability moments of resistance of the two leaves, provided they are tied in accordance with the provisions of BS 5628, clause 36.4.5. This is considered acceptable, since the resistance moment is generated by the rotation of the leaves and, once it has reached its full value, does not reduce significantly through further rotation.

Allowable flexural compressive stresses:

- outer leaf, \( P_{ubc} = \frac{1.1 \times 8.5}{2.5} \) (35 N/mm\(^2\) bricks in designation (iii) mortar)
  \[= \frac{3.74 \text{ N/mm}^2}{2} \]
- inner leaf, \( P_{ubc} = \frac{1.1 \times 5.8}{2.5} \) (20 N/mm\(^2\) bricks in designation (iii) mortar)
  \[= \frac{2.552 \text{ N/mm}^2}{2} \]

Minimum axial load in leaves:

- outer leaf = \( 0.9 \times 19 \times 0.1025 \times 2.7 \) = 4.732 kN/m,
- inner leaf = \( 4.78 + 4.732 \) (roof dead – roof uplift + ow brickwork) = 9.512 kN/m.

Minimum widths of stress blocks:

\[ w_s = \frac{P_{ubc}}{\text{axial load},} \]

- outer leaf, \( w_s = \frac{4.732 \times 10^3}{3.74 \times 1000} \) = 1.26 mm,
  \[= \frac{102.5 - 1.26}{2} \] = 50.62 mm (see Figure 39),
- inner leaf, \( w_s = \frac{2.552 \times 1000}{3.72 \text{ mm},} \]
  \[= \frac{102.5 - 3.72}{2} \] = 49.39 mm (see Figure 39).

\[ \begin{align*}
\text{outer leaf:} & \quad \text{inner leaf:} \\
102.5 \quad & \quad 102.5 \\
4.732 \text{ kN} \quad & \quad 9.512 \text{ kN} \\
50.62 \text{ mm} \quad & \quad 3.72 \text{ mm} \\
3.74 \text{ N/mm}^2 \quad & \quad 2.552 \text{ N/mm}^2 \\
1.26 \text{ mm} \quad & \quad 49.39 \text{ mm} \\
\end{align*} \]

\( M_{Res} = 4.732 \times 0.05062 \times 0.24 \text{ kN.m} \)
\( M_{Res} = 9.512 \times 0.04939 \times 0.47 \text{ kN.m} \)
\( \text{total } M_{Res} = 0.24 + 0.47 = 0.71 \text{ kN.m} \)

stability moment of resistance

26
Stability moments of resistance:
outer leaf, \( MR_s = 4.732 \times 0.05062 \)
= 0.24 kNm,
inner leaf, \( MR_s = 9.512 \times 0.04939 \)
= 0.47 kNm,
total \( MR_s = 0.24 + 0.47 \)
= 0.71 kNm.

This stability moment of resistance is now considered to provide partial fixity to the base of the wall span.

**Design bending moment**

Calculate position of maximum span moment by locating zero shear position:

shear at roof prop
\[
\frac{\gamma W h}{2} \frac{MR_s}{h} = \frac{1.4 \times 0.9 \times 2.55}{2.55} = 1.329 \text{ kN/m},
\]
point of zero shear from roof prop
\[
= 1.329
\]
maximum wall moment
\[
(1.329 \times 1.055) - (1.4 \times 0.9 \times 1.055) \]
\[
= 1.402 - 0.701 = 0.701 \text{ kNm}
\]

It is considered that, if the stability moment of resistance at the base of the wall should exceed \( \frac{\gamma W h^2}{8} \) - that of a propped cantilever - the applied moment should be limited to this value, and the wall designed as a true propped cantilever in which the maximum wall moment would be \( \frac{9\gamma W h^2}{128} \)

The bending moment diagram for this example is shown in Figure 40.

**Check stresses at level of \( M_w \)**

The stresses at the level of the maximum wall moment, \( M_w \), will be calculated using the formula

\[
\left(\frac{f_{ck}}{y_m} + g_d\right) Z,
\]

given in BS 5628, clause 36.5.3.

Hence, design moment of resistance = \( \left(\frac{f_{ck}}{y_m} + g_d\right) Z \), where:

- \( f_{ck} \) is the characteristic flexural strength (tensile),
- \( y_m \) is the partial safety factor for materials,
- \( g_d \) is the design vertical dead load per unit area,
- \( Z \) is the section modulus of wall section.

For both leaves,
\[
Z = \frac{1000 \times 102.5^2}{6} = 1.751 \times 10^4 \text{ mm}^3.
\]

For outer leaf,
\[
g_d = 0.9 \times 19 \times 1.055 = 0.018 \text{ N/mm}^2.
\]

For inner leaf,
\[
g_d = 0.018 + \frac{4.78}{102.5} = 0.0646 \text{ N/mm}^2.
\]
As, at the level of $M_w$, the two leaves deflect by the same amount and their stiffnesses are equal, the bending moment at this level will be shared equally between them. Hence, $BM$ per leaf $= 0.701 \times 0.5 = 0.35 \text{kNm}$.

Design MR of outer leaf (7% water absorption bricks)

\[
\text{in designation (iii) mortar)} = \left(\frac{0.5}{2.5} + 0.018\right) 1.751 \times 10^6 = 0.382 \text{kNm}.
\]

This is satisfactory as it exceeds the applied bending moment of 0.35 kNm.

Now calculate the characteristic flexural strength required for the inner leaf brickwork:

\[
0.35 \times 10^6 = \left(f_{\text{ks}} + \frac{0.0646}{2.5}\right) 1.751 \times 10^6,
\]

\[
\text{required } f_{\text{ks}} = 2.5 \left(\frac{0.35 \times 10^6}{1.751 \times 10^6} - 0.0646\right) = 0.338 \text{N/mm}^2.
\]

Then, from Table 3 of BS 5628, the inner leaf bricks are required to have a water absorption of between 7% and 12%, and be set in a designation (iii) mortar. This compares with the requirement for 20N/mm² bricks, set in a designation (iii) mortar, from the compressive strength part of the calculations. The final choice of bricks must balance these requirements.

**Overall stability**

In line diagram form, Figure 41 shows the main walls which will provide overall stability to the structure. The numbered walls provide stiffness to the narrow axis, whilst the lettered walls provide stiffness to the longer axis of the building. The unmarked walls, although capable of offering some resistance to the wind forces, are ignored to simplify the calculations.

Most of the walls, particularly the main crosswalls numbered 3 to 16 inclusive, intersect with other walls to form T, L, and Z-shapes on plan – thus providing extremely stiff sections to act as shear walls. However, and again for simplicity of calculation, only the straight rectangular sections of these walls will be considered as effectively resisting the lateral wind moments on the building as a whole. If this proved to be insufficient, a calculation based on the actual number and geometric shape of the shear walls would be carried out, sharing the wind load in accordance with the loaded condition and shear wall stiffness.

**Relative stiffness of shear walls**

The total wind moment acting on the building will be shared between the shear walls resisting that moment, in proportion to their stiffnesses and the load and span configurations. For simplicity, this will be considered as equating to $\frac{L^3}{\Sigma L^3}$.

| Longitudinal walls (see Figure 41): | |
|---|---|---|---|---|
| wall letter | length, L<sub>i</sub> | L<sup>3</sup> | No of leaves | $\Sigma L^3$ | wind moment share |
| B, K, N, P, Q | 2.5 | 15.6 | (x6) | 93.6 | L<sup>3</sup> / $\Sigma L^3$ |
| C, D, S, T, U, V | 3.5 | 42.9 | (x12) | 514.8 | 0.0093 |
| A, F, R, E | 4.0 | 64.0 | (x7) | 448.0 | 0.0255 |
| G, H, J, L, M | 5.0 | 125.0 | (x5) | 625.0 | 0.0381 |

\[
1681.4
\]
Latitudinal walls (see Figure 41):  

<table>
<thead>
<tr>
<th>wall number</th>
<th>length, L, (m)</th>
<th>L²</th>
<th>No of leaves</th>
<th>ΣL²</th>
<th>wind moment share</th>
</tr>
</thead>
<tbody>
<tr>
<td>1, 2, 17</td>
<td>5.0</td>
<td>125.0</td>
<td>(×5)</td>
<td>625.0</td>
<td>0.034</td>
</tr>
<tr>
<td>3 to 16 inclusive</td>
<td>6.0</td>
<td>216.0</td>
<td>(×14)</td>
<td>3024.0</td>
<td>0.059</td>
</tr>
</tbody>
</table>

Note that in calculating the relative stiffness, the external cavity walls (A, B, C, D, E, R, S, T, U, V, 1, 17) are each considered to provide two leaves of 102.5 mm brickwork to resist the wind moment.

**Design wind moments on building**

Longitudinal direction:

1.4 × 0.9 × 14 × 24.3 × 12.15 = 5208 kNm.

Latitudinal direction:

1.4 × 0.9 × 32 × 24.3 × 12.15 = 11904 kNm.

These are the maximum total wind moments acting in each direction, which the shear walls are required to resist within the lowest storey height of the building.

**Consider typical crosswall(7) in ground storey**

Share of total wind moment = 0.059 × 11904 = 702.3 kN.m.

Minimum dead load in wall = (0.9 × 210) + (0.9 × 19 × 0.1025 × 24.3) = 231.6 kN/m².

Therefore, \( g_d \) = \( \frac{231.6 \times 10^3}{102.5 \times 1000} \) = \( 2.259 \) N/mm²,

\( \text{and } Z = \frac{102.5 \times 6000^2}{6} = 6.15 \times 10^9 \) mm².

Design moment of resistance = \( \left( \frac{f_{ks}}{f_m} + g_d \right) Z \)

\[ = \left( \frac{0.5}{2.5} \times 2.259 \right) 6.15 \times 10^9 \]

\[ = 15123 \text{ kNm at base of wall.} \]

Which exceeds the applied wind moment of 702.3 kN.m, thus justifying the simplification of the calculations, made earlier, in assessing the relative stiffnesses.

This design moment of resistance has been calculated on the assumption that full restraint against buckling is provided by the ground floor slab and, therefore, no stress reduction factors are applicable – which is a perfectly reasonable proposition. However, at mid-storey height, the buckling tendency of the length of shear wall under flexural compressive stress can have an effect on the allowable stresses, and the design method described earlier in 'Stability of shear walls' will be applied as follows:

allowable flexural compressive stress, \( f_{ubc} = \frac{1.1f_k}{Y_m} \)

\[ = \frac{1.1 \times 0.749 \times 12.2}{2.5} \]

\[ = 4.021 \text{ N/mm}^2 \]

design flexural compressive stress, \( f_{ubc} = \frac{231.6 \times 10^3 + 702.3 \times 10^6}{102.5 \times 6000 + 6.15 \times 10^9} \]

\[ = 0.38 + 0.114 \]

\[ = 0.494 \text{ N/mm}^2 \]

Other shear walls providing overall stability to the building should be checked using the same basic principles but, by inspection, these also should prove to be comfortably within the allowable stresses given in BS 5628.

**Accidental damage design**

The performance parameters for designing buildings to withstand accidental damage are set out in section 5 of BS 5628, and under Building Regulation D17. Interpretation of the wording of both these documents can be complex and conflicting. The main objective of the rules for accidental damage design is to produce a sufficiently robust structure to withstand damage of limited proportions. Hence, the design techniques should be a combination of stress calculations and descriptive reasoning to demonstrate, beyond reasonable doubt, the inherent robustness of the structure under various damage situations.

Loadbearing brickwork crosswall construction
This building, being over 5 storeys in height, is classed as a Category 2 building. Table 12 of BS 5628 gives three options for designing and detailing such a structure for accidental damage. Option 1, in which vertical and horizontal elements (unless ‘protected’) are to be proved removable, one at a time, without causing collapse, is considered most appropriate for this type of building.

**Crosswalls removed**

The normal span of the in-situ concrete floor is perpendicular to the line of the crosswalls. If a crosswall is removed, the slab is designed to span in the opposite direction onto the corridor and elevational walls using, if necessary, increased distribution reinforcement designed on the appropriately reduced partial safety factors. In addition, the slab would tend to hang, in a catenary, between the crosswalls either side of the removed wall.

Calculations could be prepared for either of these alternative means of support, and should be accompanied by a commentary on the assumptions made for the design method chosen.

**Gable wall removed**

BS 5628 states that for walls without vertical lateral supports, the whole length of external walls must be considered removable, whilst for similar internal walls, only 2.25h need be considered as the removable length. The Building Regulations do not differentiate between internal and external walls, but limit the removable length to 2.25h for all walls. It seems to the authors particularly harsh to consider, say, in a spine wall structure of 30 m or more in length, the possibility of an incident capable of removing such a disproportionate length of external wall. It is suggested, therefore, that in certain circumstances the designer should use his discretion in assessing a realistic but reasonable length for removal.

Having assessed the removable length of gable wall, consideration can now be given to the alternative means of support for the structure, following its removal. If the length removed is not excessive, consideration may be given to composite action of the masonry over acting with the floor slab immediately above the removed length of wall. This, together with the arching effect of the masonry to spread the loads over to the other side of the removed length of wall, may be all that is necessary, with the additional reinforcement, if any, being added peripherally in the in-situ floor slab.

A more complex analysis might consider two adjacent floor slabs acting as the flanges of deep I-beams with the corridor and elevational walls between them acting as the webs of the same beam. These composite sections may be used to cantilever from the last crosswall and could support, at the end of the cantilever, a similar I-shaped composite beam utilising the gable wall as the web. Thus, a framework of composite beams is provided, and reinforced accordingly, to support the structure over (see Figure 42).

![Image](image.png)

It may well be that, in the lower storeys of a multi-storey loadbearing brick structure, there is enough compressive load from above to enable the walls to be designed to withstand the lateral force of 34 kN/m², thus defining that wall as a ‘protected’ member which does not have to be considered removable.

The relative simplicity with which the requirements for accidental damage can be met in loadbearing brickwork design is indicative of the general robustness of this form of construction. This robustness was dramatically demonstrated during the last war, when numerous masonry structures, with alarming portions and corners blown out through bomb damage, remained stable. Many were simply strengthened locally to continue their useful life.
COMMERCIAL OFFICE DEVELOPMENT 4-STOREYS HIGH

Building geometry (see Figures 43 & 44)

- Overall height = 13.6 m
- Overall length = 46.0 m
- Overall width = 46.0 m
- Floor to floor height = 3.2 m
- Span of precast floors = 7.0 m

Assume: masonry density = 19.0 kN/m$^3$
- precast floors = 2.75 kN/m$^2$.
Roof and floor slabs are 225 mm thick prestressed concrete units. External facing bricks selected have a water absorption of 6.5% and a compressive strength of 50 N/mm², and are to be set in a designation (ii) mortar. Extensive quality control and testing of materials will be exercised throughout, and strict supervision will be permanently employed.

**Characteristic loads**

| Roof | dead load, G_k, PC units | = 2.75 |
|      | screed to falls           | = 1.25 |
|      | imposed load, Q_k (with direct access) | = 1.50 kN/m² |

| Floors | dead load, G_k, PC units | = 2.75 |
|        | partitions              | = 1.85 |
|        | finishes                | = 1.35 |
|        | services                | = 0.30 |
|        | imposed load, Q_k (offices) | = 2.50 kN/m² |

**Wind loading**

Is assumed to have been calculated on the basis of CP 3: Chapter V: Part 2: 1972, to give a maximum characteristic wind pressure on the walls of + 0.70 kN/m², and a maximum gross characteristic wind uplift on the flat roof of + 1.05 kN/m².

**Design of typical internal crosswall**

<table>
<thead>
<tr>
<th>Characteristic dead loads</th>
<th>Characteristic imposed load</th>
<th>Imposed load less reductions</th>
</tr>
</thead>
<tbody>
<tr>
<td>floors &amp; roof</td>
<td></td>
<td></td>
</tr>
<tr>
<td>roof</td>
<td>(7 x 4)</td>
<td>28.00 (7 x 1.5)</td>
</tr>
<tr>
<td>third</td>
<td>28 + (7 x 6.25)</td>
<td>115.50 28.0 + 17.5</td>
</tr>
<tr>
<td>second</td>
<td>71.75 + 43.75</td>
<td>159.25 45.5 + 17.5</td>
</tr>
<tr>
<td>first</td>
<td>115.5 + 43.75</td>
<td>222.95</td>
</tr>
</tbody>
</table>

With a floor to floor height of 3.20 m, a half brick thick wall would be approaching the limit of maximum slenderness ratio, and would therefore require a relatively high strength brick. In addition, a bearing width of 102 mm is not adequate to receive long-span prestressed concrete units from either side. Hence, 215 mm thick walls will be adopted for the main loadbearing walls.

Design wall in the lowest storey for the loading combination of dead plus imposed. Thus, the partial safety factors will be:

\[
\gamma_f, \text{ dead load} = 0.9 G_k \text{ or } 1.4 G_k
\]

and imposed load = 1.6 Q_k (BS 5628, clause 22).

**Design loads**

<table>
<thead>
<tr>
<th>dead</th>
<th>imposed</th>
<th>masonry</th>
</tr>
</thead>
<tbody>
<tr>
<td>159.25 \times 1.4</td>
<td>44.10 \times 1.6</td>
<td>0.215 \times 19 \times 12.8 \times 1.4</td>
</tr>
</tbody>
</table>

| total n_w | 366.71 kN/m. |

**Capacity reduction factor**

As for the previous design example, the concrete floor slabs can be assumed to provide enhanced resistance to lateral movement. Hence:
effective height \( = 0.75 \times \) clear height 
therefore, \( h_{eff} = (3.20 - 0.225) \times 0.75 = 2.23 \)
slenderness ratio \( \frac{h_{eff}}{t_{eff}} = \frac{2.23}{0.215} = 10.4 \).

The calculation of the eccentricity of the load is carried out in a similar manner to the previous design example, and from Figure 45 the following values can be calculated:

![Figure 45: Calculation of eccentricity of load](image)

\[ R_1 = 40.425 \text{ kN} \]
\[ R_2 = 19.69 \text{ kN} \]
minimum load in wall above first floor = 139.24 kN
resultant position from left hand face = 100 mm
eccentricity about \( \ell \) at top of wall = 7.5 mm,
therefore,
\[ e_\ell = 0.035t \] (which is less than 0.05t).
Thus, from BS 5628, Table 7:
with slenderness ratio = 10.4
and \( e_\ell = 0.035t \)
\[ \beta = 0.962. \]

**Partial safety factor for material strength**
Similar conditions exist to those applicable in the previous design example. Hence, from BS 5628, Table 4:
\[ \gamma_m = 2.5. \]

**Masonry strength required**
Characteristic strength of masonry required, \( f_{km} = \frac{n_w \gamma_m}{\beta t} \)
\[ = \frac{366.71 \times 2.5 \times 10^3}{0.962 \times 215 \times 1000} \]
\[ = 4.43 \text{ N/mm}^2. \]
Use bricks with a compressive strength of 15 N/mm², set in a designation (iv) mortar.

**Design of external cavity wall for wind**
The design principles will be similar to those used in the previous design example. Consideration will also be given to the walls running parallel to the span of the pc units and, for these, a 215 mm thick inner leaf will be checked. In both cases, the wall in the topmost storey will be designed.

**Case 1, wall supporting roof units**

**Consider roof uplift**
minimum roof dead load \( = 0.9 \times 4.0 \) \( = 3.60 \text{ kN/m}^2 \)
maximum wind uplift on roof \( = 1.4 \times 1.05 \) \( = 1.47 \text{ kN/m}^2 \)
nett roof dead load (after uplift) \( = 3.60 - 1.47 \) \( = 2.13 \text{ kN/m}^2 \)
nett roof dead load on wall \( = 2.13 \times 3.5 \) \( = 7.455 \text{ kN/m}^2 \)
Calculate stability moment of resistance
Assume 102.5 thick brickwork for both leaves. Allowable flexural compressive stresses:

outer leaf, \( p_{\text{abc}} = \frac{1.10 \times 10.60}{2.5} \) = 4.664 N/mm²

inner leaf, \( p_{\text{abc}} = \frac{1.10 \times 4.40}{2.5} \) = 1.936 N/mm²

Minimum axial loads in leaves:

outer leaf = 0.9 \times 0.1025 \times 19 \times 4.0 = 7.011 kN/m

inner leaf = 7.011 + 7.455 = 14.466 kN/m

Minimum width of stress blocks:

outer leaf, \( w_s = \frac{7.011 \times 10^3}{4.664 \times 1000} \) = 1.5 mm

lever arm = \( \frac{102.5 - 1.5}{2} \) = 50.5 mm

inner leaf, \( w_s = \frac{14.466 \times 10^3}{1.936 \times 1000} \) = 7.5 mm

lever arm = \( \frac{102.5 - 7.5}{2} \) = 47.5 mm

Stability moment of resistance

outer leaf = 7.011 \times 0.0505 = 0.354

inner leaf = 14.466 \times 0.0475 = 0.687

total MRₗ = 1.041 kNm

The stability moment of resistance is shown diagrammatically in Figure 46.

This stability moment of resistance is less than the design bending moment at the base of the wall which would be applicable to a true propped cantilever. Hence, the stability moment of resistance is considered as a partially fixed base, and the maximum design bending moment in the height of the wall is calculated to coincide with the level of zero shear.

Maximum wall moment

Shear at roof prop \[ \frac{\gamma_t W_k h}{2} - \frac{M_{R}_{\text{h}}}{h} = \frac{1.4 \times 0.7 \times 3.2}{1.268} - 3.2 = 1.243 \text{ kN/m}. \]

Point of zero shear from roof prop \[ = \frac{1.4 \times 0.7}{1.268} = 1.268 \text{ m}. \]

Maximum wall moment, \( M_w = (1.243 \times 1.268) - (1.4 \times 0.7 \times \frac{1.268^3}{2}) \]
\[ = 1.576 - 0.788 = 0.788 \text{ kNm}. \]

The design bending moment diagram is shown in Figure 47.
The two leaves of the cavity wall deflect, under wind load, by the same amount and are of equal stiffness. Therefore, the maximum wall moment will be shared equally between two leaves. Thus, $M_w \text{ per leaf}=0.394 \text{ kNm}$.

Now calculate the characteristic strengths required for the brickwork in the two leaves to withstand this design bending moment.

From $f_{ks}$ required $= \left( \frac{M_w}{Z} - g_d \right)$, as in the previous design example:

Outer leaf, $Z = \frac{1000 \times 102.5^2}{6} = 1.751 \times 10^6 \text{ mm}^4$  
$g_d = 0.9 \times 19 \times 2.068 = 0.035 \text{ N/mm}^2$  

Inner leaf, $Z = \text{as above} = 1.751 \times 10^6 \text{ mm}^4$  
$g_d = 0.035 + \frac{7.455}{102.5} = 0.108 \text{ N/mm}^2$.

**Characteristic strengths required**

Outer leaf, $f_{ks}$ required $= \left( \frac{0.394 \times 10^6}{1.751 \times 10^6} - 0.035 \right) 2.5$  
$= 0.475 \text{ N/mm}^2$.  

Which is less than the $f_{ks}$ provided of 0.5 N/mm² for the facing bricks and mortar specified.

Inner leaf, $f_{ks}$ required $= \left( \frac{0.394 \times 10^6}{1.751 \times 10^6} - 0.108 \right) 2.5$  
$= 0.292 \text{ N/mm}^2$.

Therefore, use bricks with a water absorption of between 7% and 12% set in a designation (iv) mortar for the inner leaf.

**Case 2, check the non-loadbearing external walls**

**Stability moment of resistance (215 thick inner leaf)**

Allowable flexural compressive stresses, as calculated for loadbearing external wall.

Minimum axial load in leaves:

- outer leaf $= \text{as for previous wall} = 7.011 \text{ kN/m}$
- inner leaf $= 0.9 \times 0.215 \times 19 \times 4.0 = 14.706 \text{ kN/m}$

Minimum widths of stress blocks

- outer leaf, $w_s = \text{as for previous wall} = 1.5 \text{ mm}$
- lever arm $= 50.5 \text{ mm}$
- inner leaf, $w_s = \frac{14.706 \times 10^3}{1.936 \times 1000} = 7.6 \text{ mm}$
- lever arm $= \frac{215 - 7.6}{2} = 103.7 \text{ mm}$

**Stability moments of resistance**

- outer leaf $= 7.011 \times 0.0505 = 0.354$
- inner leaf $= 14.706 \times 0.1037 = 1.525$
- total $MR_s \text{ kNm}$

The stability moment of resistance exceeds the applied design bending moment applicable to a true propped cantilever, which is calculated as:

$\frac{\gamma_r W_n h^2}{8} = \frac{1.4 \times 0.7 \times 3.2^2}{8} = 1.254 \text{ kNm}$.

Hence, the wall will be designed as a propped cantilever, and the condition at $\frac{3}{8}h$ down from the roof will be examined.

**Maximum wall moments**

$M_w = \frac{9 \gamma_r W_n h^2}{128} = \frac{9 \times 1.4 \times 0.7 \times 3.2^2}{128} = 0.706 \text{ kNm}$.

The two leaves will deflect equally under wind loading, but they are not of equal stiffness, hence the maximum wall moment will be shared between the two leaves in proportion to their stiffnesses.

Thus:

- outer leaf, $1 = \frac{1000 \times 102.5^2}{12} = 89.7 \times 10^4 \text{ mm}^4$

Loadbearing brickwork crosswall construction
inner leaf, \( I = \frac{1000 \times 215^2}{12} = 828.2 \times 10^6 \text{ mm}^4 \)

Therefore, share of \( M_o \)

outer leaf, \( M_o = 0.706 \left( \frac{89.7}{89.7 + 828.2} \right) = 0.069 \text{ kNm} \)

inner leaf, \( M_o = 0.706 \left( \frac{828.2}{89.7 + 828.2} \right) = 0.637 \text{ kNm} \)

**Characteristic strengths required**

From:

required \( f_{\text{ks}} = \left( \frac{M_o}{Z} - g_d \right) \gamma_m \)

outer leaf, \( Z = \frac{1000 \times 102.5^2}{6} = 1.751 \times 10^6 \text{ mm}^2 \)

\( g_d = 0.9 \times 19 \times 2.068 = 0.0354 \text{ N/mm}^2 \)

inner leaf, \( Z = \frac{1000 \times 215^2}{6} = 7.70 \times 10^6 \text{ mm}^2 \)

\( g_d = 0.9 \times 19 \times 2.068 = 0.0354 \text{ N/mm}^2 \)

Therefore:

outer leaf \( f_{\text{ks}} \) required = \( \left( \frac{0.069 \times 10^6}{1.751 \times 10^6} - 0.0354 \right) \times 2.5 \)

= 0.010 \text{ N/mm}^2

inner leaf \( f_{\text{ks}} \) required = \( \left( \frac{0.637 \times 10^6}{7.70 \times 10^6} - 0.0354 \right) \times 2.5 \)

= 0.12 \text{ N/mm}^2

Each of these required values is less than those calculated for the loadbearing external walls, hence the same brickwork specifications will be adequate.

In each of these external wall designs, consideration must also be given to the effect of window openings, and for a suggested design method for dealing with such a perforated wall, readers are referred to Structural Masonry Designers’ Manual.

The requirements for supporting the outer leaf of cavity walls in tall buildings have been discussed earlier under External Walls. Whilst this structure is of four storeys only it is 13.2 m high. Thus, a support is necessary, and it should preferably be introduced at second floor level.

**Overall stability**

By considering the plan shape of the whole building, and the disposition of the substantial crosswalls and corridor spine walls, it can be stated that ‘by inspection’ the overall stability of the building does not require to be justified by calculation. The previous design example, on a very much more slender structure employing narrower crosswalls, demonstrated that the stresses in the shear walls resisting lateral wind loading were relatively insignificant.

**Accidental damage**

The building is four storeys high and, therefore, does not come under the requirements of D17 of the Building Regulations. As such, the building does not require any additional precautions to be taken with regard to accidental damage, other than those generally provided for in clause 2 of BS 5628 and summarised earlier in this publication. Tying of the adjacent spans of precast concrete units, as shown in Figures 3 & 4, can provide an effective and inexpensive means of alternative support in a collapse situation, and is, perhaps, the least that the designer may consider to be warranted.

**Other applications**

The building considered has been described as an office building, providing a number of adequately sized rooms between the widely-spaced crosswalls. The design would require little alteration to suit the functions of a school classroom building, in which the room sizes are compatible, and only a small increase in the imposed load, with possibly a compensating reduction in the partitions load, being all that is necessary.

Similarly, small four or six bed wards in modern hospital developments would be admirably suited to this form of construction, with the application of an almost identical design process.
Above NURSES' HOSTEL, OXFORD STREET
MATERNITY UNIT, LIVERPOOL
Twin six-storey units with in situ concrete floors carried on half-brick thick crosswalls. Provisions for vertical expansion incorporated in external walls.

Architects Ormrod & Partners
Structural engineers W. G. Curtin & Partners.

Back cover BLAIR COURT, BIRKENHEAD
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