DESIGN OF FREE STANDING WALLS
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The author acknowledges the assistance of Dr. K. Al-Hashimi CEng MICE PhD MBGS and Robert G. D. Brown MA CEng MICE in the preparation of the draft of this document.

**Editor’s Note:**

Apart from a few minor amendments, this 1984 edition is essentially a reprint of the 1983 edition. In view of the impending publication of BS 5628: Part 2: Reinforced & Prestressed Masonry, designers should note that Section 4 on reinforced brickwork uses Special Publication 91\(^{(12)}\) as the basis of design for the limit state method. This section will be revised to incorporate the latest design guidance when BS 5628: Part 2 is published.

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Price £7.00
Introduction

The Design of Freestanding Walls DG12 was first published by BDA in February 1983 and subsequently revised and reprinted in February 1984. Due to its popularity with designers and because much of the guidance is still relevant, DG12 is being retained in the BDA's Publications List.

Codes, Standards and other reference material have changed during the intervening 12 years since the guide was reprinted in 1984, and the purpose of these information notes is to highlight those important reference changes which have a significant influence on the use of the guide. These updated references will need to be fully considered by designers in the interpretation and use of DG12.

It is not possible to issue a textural line-by-line addendum sheet to the guide and no attempt is being made to do this. In due course when all the essential Codes, Standards and other reference sources which are currently changing have come into effect, it is the Association's intention to completely revise DG12.

Design for Exposure/Durability

Pages 4 to 8 inclusive of DG12 provide advice on exposure, durability and associated detailing considerations for the design of freestanding walls. Some of this guidance has been superseded by more up-to-date information and reference material. The "special" and "ordinary" qualities classification of clay bricks is now redundant and has been superseded.

Reference should be made to BS 3921(1), BS 187(2), BS 4729(3) and BS 6649(13) and to BS 5628:Use of Masonry:Part 3:1985:Materials and components, design and workmanship(14). Those clauses of BS 5628:Part 3:1985 which are of particular relevance to brickwork masonry are: Clause 21 - Exclusion of Moisture (sub-clauses 21.2 and 21.7), Clause 22 - Durability, Clause 23 - Selection of Mortars, and Table 13 - Durability of Masonry in Finished Construction.

A further source of information regarding durability of brick masonry construction is BDA Design Note 7 - Brickwork Durability(15).

Wind Loads

DG12 derives wind loadings in accordance with BS CP3:Chapter V:Part 2:1972:Wind Loads. At the present time this Code is still relevant to wind loading derivation, although the current version has been amended from the 1972 edition. Consequently figure 3 on page 11 of DG12 - Basic wind speed in m/s requires revising in accordance with the latest amendment of CP3.

During 1995 it is expected that BS 6399:Loading for Buildings:Part 2:1995:Code of practice for wind loads, will be published by the British Standards Institution. This new wind load code will be a complete revision of CP3. BS 6399:Part 2 is likely to coexist alongside CP3:Chapter V:Part 2 for about 12 months, after which time CP3 will be withdrawn. BS 6399:Part 2 will contain specific information for the derivation of wind loads on freestanding walls.

DG12 can be used in either the context of CP3:Chapter V:Part 2 or BS 6399:Part 2. If wind loading derivation is in accordance with BS 6399:Part 2 then Table 2 (page 12) of DG12 becomes redundant whilst Figure 7 (page 18) of DG12 continues to be relevant if "p", the design wind pressure is derived fully in accordance with BS 6399:Part 2 and incorporates the appropriate partial factor of safety for loading (γf) from BS 5628 (usually γf = 1.2). This latter requirement also applies to DG12 guidance in respect of the use of "p", the design wind pressure, referenced on pages 17, 18, 19, 20, 21 and 23 for unreinforced brickwork design and to similar design wind pressure references for reinforced brickwork, (pages 26 to 34 inclusive of DG12).

Structural Masonry Design Codes

All references in DG12 to BS 5628:Use of Masonry: Part 1:1978:Structural use of unreinforced masonry, should be replaced by reference to BS 5628:Use of Masonry:Part 1:1992:Structural use of unreinforced masonry. For brickwork freestanding wall design to DG12 the 1992 version of BS 5628:Part 1 is identical to the 1978 edition with the exception that the characteristic shear strength of unreinforced masonry in designation iii) mortar is reduced to 0.15 N/mm² (from 0.35 N/mm²). Page 21 of DG12 shows a shear strength calculation for piers and if the revised characteristic shear strength...
of 0.15 N/mm² is taken into account the pier design demonstrated remains adequate.

Pages 26 to 34 inclusive of DG12 demonstrate reinforced brickwork design using two alternative methods. The first is a permissible stress design method based on CP 111:Part 2:1970, while the second is limit states based to SP91. Reference to SP91 should be replaced by BS 5628:Use of Masonry:Part 2:1985: Structural use of reinforced and prestressed masonry. BS 5628:Part 2 became available after DG12 was published. The detailed design approach and worked examples for limit states design shown in DG12 from pages 29 to 34 inclusive need to be modified in respect of the recommendations of BS 5628:Part 2:1985, although the basic principles of reinforced masonry design using this Code remain the same as those given in SP91. SP91 was a forerunner to BS 5628:Part 2.

An amended version of BS 5628:Part 2 is expected to be published in the Autumn of 1995. Upon publication by the British Standards Institution this Code should be substituted for the 1985 edition. Major changes will include guidance on design for durability of reinforcing steels (cover and infill concrete quality) and changes to design methods for laterally loaded masonry panels incorporating bed-joint reinforcement. References will also be brought fully up-to-date.

References
The list of references on page 35 of DG12 should be amended as follows (others not amended remain current):


(4) Delete reference to BS 3798:1964 which is a withdrawn standard and add in its place:


Note that in due course CP3:Chapter V:Part 2 is likely to be withdrawn.


Add new references:


The following two references may be of interest and are published by BRE. They are compatible with DG12 and its recommendations.


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Design of free standing walls

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The Brick Development Association
INTRODUCTION

This document, which has been prepared for the guidance of civil and structural engineers, architects and builders, deals with the design and use of plain and reinforced free-standing brick walls not forming part of a building.

Free-standing brick walls are widely used for boundary demarcation, landscaping, screening, security, and noise barriers. When properly constructed, free-standing brick walls have proved to be extremely durable and pleasing in appearance. There are many examples of brick walls, hundreds of years old, which apart from occasional pointing and attention to coping require no other maintenance. By modern standards, most of these old walls, built by rule of thumb, are clearly over-designed.

However, since the turn of the century, and particularly after the Second World War, financial constraints and the phenomenal growth of mass housing estates produced some very unsatisfactory examples of free-standing walling. Strange as it may seem, walls are not subjected to statutory control, except in the Inner London Area where approval is required when the height exceeds 1.83 m (6 ft) above ground level. It is, therefore, not surprising that in the majority of cases walls are put up rather than designed, and all too frequently prematurely deteriorate or blow over.

In most cases, a durable and stable wall will cost little more than a sub-standard one. All that is required is the selection of suitable materials combined with an efficient arrangement of brickwork.

1 & 2 Free-standing wall screening a small mews development and reducing noise from a very busy trunk road.
Staggered boundary wall enhances the roadside.

Design of free standing walls
EXPOSURE
More often than not, free-standing walls are exposed to the full effects of the weather. The action of wind, as a lateral force, is catered for by the strength design. But, the combined effect of rain and driving wind requires the use of suitable materials and correct constructional details.

Figure 1 shows the driving rain index map of the United Kingdom which, at present, is the best published guide to the severity of weather conditions. In general, bricks for free-standing walls in the moderate and sheltered zones should comply with the frost resistance requirements (but not necessarily the other requirements) for special quality bricks. However, where the wall is properly weathered (i.e., provided with an overhanging coping) ordinary quality bricks may be used if recommended by the manufacturers for the particular circumstances. In areas of severe exposure, special quality bricks to BS 3921\(^{(1)}\), or other bricks having a high frost resistance, are recommended. Calcium silicate bricks Class 3 or stronger, in accordance with BS 187,\(^{(2)}\) are suitable for all degrees of exposure.

These are very general guidelines which may be modified in the light of local conditions. Users should contact the manufacturers to confirm that their bricks are suitable for the intended degree of exposure.

It is also well worth while to examine existing walls in the neighbourhood and note the performance of particular bricks after a few winters’ service. The exposure of saturated bricks to frost or, even more importantly, to the freeze/thaw cycle over several winters gives a very useful guide to durability.

Some of the more economical walls recommended in this guide incorporate half-brick panels. These walls are particularly vulnerable to saturation in the winter months, but should perform satisfactorily provided they are constructed in accordance with the advice given in this guide and under competent supervision.

COPINGS AND CAPPINGS
The function of copings and cappings is aesthetic as well as protective. The appearance of a wall may be dramatically changed by altering the coping or capping.

The protective function is against vertical water penetration and in shedding rainwater clear of the face as effectively as possible. The ideal coping should be durable, waterproof, weather the brickwork, and constructed preferably with interlocking units. The overhang should include a throating recess or a drip not less than 13 mm wide, with the outer drip edge at least 40 mm from the face of the brickwork as shown in Figure 2c. Different shapes and types of copings and cappings are shown in Figure 2.

For cappings of special brick shapes, reference should be made to BS 4729,\(^{(3)}\) and for copings of clayware, concrete or stone, to BS 3798\(^{(4)}\). A minimum weight of 1.5 kN/m² is preferred for copings with concrete or stone units. Some brick manufacturers can supply their own capping system which incorporates a dpc.

In many instances, for aesthetic or functional reasons, it is necessary to use a brick-on-edge capping. Such a capping will perform reasonably well, provided that it is constructed in accordance with the recommendations of Table 1. Suitable galvanised steel, stainless steel or non-ferrous metal anchors...
Driving rain index

Design of free standing walls
may be used to prevent movements of end units, Figure 2n. For best results an overhang such as shown in Figure 2 (f-i) should be provided.

**MORTARS**

Free-standing walls derive part of their resistance to lateral forces from their flexural strength. Where adhesion between bricks and mortar is not achieved, or is lost due to deterioration of the mortar, the strength may be reduced by as much as 50%. It is therefore imperative to ensure permanent flexural tensile adhesion between the bricks and the mortar.

There is some evidence that the presence of entrained air bubbles, due to the use of plasticisers, results in an inferior adhesion. Experience shows that plasticised or masonry cement mortars may be perfectly workable when the cement content is as low as a half or a third of that specified. Thus, a visual check on mortar workability cannot indicate whether it has been correctly gauged.

The following recommendations should ensure adequate and durable adhesion between bricks and mortar:

1. Cement/lime/sand or cement/sand mortars should be used. Plasticised or masonry cement mortars are not recommended, but may be used with the permission of the engineer and under close supervision aimed at ensuring that:
   a. the mortar is correctly gauged with adequate water content
   b. mixing is strictly controlled to prevent excessive air entrainment
   c. partially set mortar (normally after two hours) is discarded and not reconstituted
   d. the suction rate of units is adjusted by controlled wetting.

In general, air-entrained mortars are much more prone to site abuse, and circumstances may arise where adequate adhesion is not achieved.

2. In dry, hot weather, bricks should be wetted by lightly spraying stacks or docking individual units. Alternatively, a wetter mortar may be used.

3. Where bricks contain sulphates in excess of 0.5% (ie, where bricks do not conform to the maximum allowable soluble sulphate content for special quality bricks to BS 3921) sulphate resisting Portland
brickwork capping above dpc to be special quality in 1:4:3 mortar, or class 4 calcium silicate bricks in 1:6:4½ mortar. See text and table 1 for wall and foundations brickwork and movement joint requirements.
Table 1  Brick and mortar requirements and associated materials

<table>
<thead>
<tr>
<th>Position in wall</th>
<th>Bricks</th>
<th>Mortar</th>
<th>DPC</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foundations to 150 mm above ground level</td>
<td>Clay to BS3921 (frost resistant), Calcium Silicate Class 3 to BS187</td>
<td>1:4:3 SRPC if sulphates in brick or soil, otherwise 1:2:3 PC Where in doubt sulphate resisting cement should be used</td>
<td>Not essential with low absorption bricks, but if desired provide two courses of bricks having absorption of not more than 7% or two courses of slates to BS743 fully half-lapped and bedded in mortar</td>
<td>No other form of dpc should be used, unless allowed for in structural design.</td>
</tr>
</tbody>
</table>

From 150 mm above ground level to u/s of coping | Clay to BS3921 | Minimum 1:1:6 or 1:4:4 SRPC if sulphates in bricks in excess of 0.5% otherwise 1:1:6 or 1:4:4 PC | 1:4:3 and 1:4:4 are more durable |

| Coping or Capping | Clay Special Quality to BS3921 | 1:4:3 PC | Flexible dpc to BS 473 below coping or capping. Preferably Permagrip | To improve waterproofing of coping aluminium stearate additive may be used |

Calcium Silicate Class 3 to BS187 |

1:1:6 PC |

Calcium Silicate Class 4 to BS187 |

1:4:4 PC |

Cement (SRPC) should be specified and the mortar strength must not be weaker than 1:1:6. In cases of near maximum permitted sulphate content and/or severe exposure 1:1:4, or even 1:1:3 SRPC, is preferable – see Table 1.

DAMP PROOF COURSES

The strength of unreinforced free-standing walls described in this guide is dependent on their flexural tensile stress. Consequently, the use of any dpc near ground level incapable of providing the necessary adhesion across the joint is not permitted unless the ‘No Tension’ method of design is used, as explained in ‘Methods of Design’ (page 20).

In practice, the only dpcs which meet this requirement are:
(a) two courses of (engineering) bricks to BS 3921 or BS 743(as). For free-standing walls, special quality bricks with a water absorption of not more than 7% are considered to be adequate;
(b) two courses of slates fully half lapped and bedded in mortar to BS 743.

In both cases, 1:4:3 PC or SRPC mortar (as appropriate) should be used. However, in many cases where walls are constructed of frost-resistant bricks, the need for a dpc near ground level is questionable.

Below the coping or capping, a suitable flexible dpc should be provided, fully bedded on the underside and on top in a suitable mortar (see Table 1) and projecting 10 to 15 mm from the face to form a drip. Permagrip dpc is particularly suitable for this purpose because of its superior adhesion to mortar. Flexible dpc’s must never be cut back or pointed over.

MOVEMENT JOINTS

In common with other building materials, brickwork is not dimensionally stable, and both reversible and irreversible changes occur. In the case of free standing walls, the following main factors must be taken into account in deciding the frequency of movement joints:

1. moisture expansion of clay bricks
2. thermal expansion or contraction
3. drying shrinkage, applicable in practice to calcium silicate bricks only.

Movement joints should be constructed so that, in effect, they introduce a complete separation over the
4, 5 & 6 Correctly designed movement joints.

7 & 8 Cappings carried across movement joints. Though a newly constructed wall, the beginnings of potential problems are already apparent.
full height of the wall, including the dpc and coping or capping but not the foundations. Detailed recommendations for the design of movement joints are given in Section 3.

FOUNDATIONS
The foundation requirements for free-standing walls may be less onerous than those for buildings.

It is suggested that, for free-standing walls not forming part of a building and not exceeding 2.5 m in height, a foundation depth of 0.5 m, on a sufficiently firm bottom in any type of soil, provides a reasonable compromise between cost considerations and stability. For higher walls in cohesive soils, a minimum of 0.75 m is recommended. Where mixed made-up ground is present, it would be reasonable to assume an allowable net bearing pressure of 50 kN/m², provided no organic soil is present and a reasonable degree of consolidation has been attained over the years.

Fresh fill is not suitable for any foundations unless mechanically compacted in thin layers. In such conditions, the allowable net bearing pressure should, however, be limited to 25 kN/m² and the footings should be reinforced.

For reasons explained on page 17, the depth of brickwork below ground ought not to exceed 200 mm. Detailed recommendations in respect of foundations for various walls, and worked examples, are provided in Section 3.

WIND LOADS
The design wind speed $V_\text{w}$ is calculated on the basis of CP 3, Chapter V, Part 2:

$$V_\text{w} = V_S S_1 S_2$$

where

- $V =$ basic wind speed in m/sec (see Figure 3)
- $S_1 =$ topographic factor, normally taken as 1
- $S_2 =$ ground roughness, size of structure and height factor. 0.6 value applies to walls of any length not exceeding 3 m height above ground level, located in the country with many wind breaks, small towns or outskirts of large cities.
- $S_3 =$ statistical factor taken as 1 in all circumstances.

Therefore,$V_\text{w} = 0.6V$ m/sec.

Since the limit state approach is adopted, the design wind speed is redesignated $V_\text{c}$, the characteristic wind speed, but note that $V_\text{c} = V_\text{s} = 0.6V$. The design wind pressure can now be calculated:

$$p = KV_\text{c}^2 C_\text{f} \gamma_r N/m^2$$

where

- $p =$ design wind pressure, N/m²
- $K =$ 0.613 universal coefficient from the wind code
- $C_\text{f} =$ force coefficient, assumed 1.2
- $\gamma_r =$ partial safety factor for laterally loaded free-standing walls = 1.2.

Therefore,$p = 0.613 \times (0.6V)^2 \times 1.2 \times 1.2$

$$= 0.318V^2 N/m^2$$

For other $S_1$ and $S_2$ factors, the formula may be written in a more general form:

$$p = 0.88 (S_1 \times S_2 \times V)^2$$

No guidance is given in CP 3, Chapter V, Part 2 concerning the appropriate force coefficient for free-standing walls but, in the Wind loading handbook, the authors suggest that such walls should be treated as members of an unclad structure. A table is provided in the handbook, covering different cases and showing the force coefficient varying between 1.2 and 2.

However, if the cases shown are related to the actual configuration of boundary walls, it becomes clear that a coefficient of less than 2 hardly ever applies. The contention that a force coefficient of 2 is exceptionally high, and does not accord with the history of service behaviour of garden walls, may be best illustrated by the fact that, until 1973, such walls in the area covered by the GLC By-Laws were designed to 6lb/ft² (290 N/m²). The current practice in the GLC is to use $C_\text{f} = 1.2$ which produces 8 lb/ft² (380 N/m²), a 33% increase. But, if $C_\text{f} = 2$, as suggested by the handbook, it is assumed, the walls will have to be designed to withstand 13 lb/ft² (636 N/m²) - undoubtedly a very unreasonable proposition.

The formula derived above, $p = 0.318V^2$, applies to ground roughness Category 3 which is defined as: 'Country with many windbreaks (hedges, copses, orchards, rows of trees, or similar), small towns or outskirts of large cities'. This formula is likely to be appropriate in the majority of cases but, in the
Basic wind speeds in m/s
open country with only scattered wind breaks (Category 2), equation \( p = 0.396V^2 \) should be used. Where there are no windbreaks at all in open country (Category 1), \( p = 0.537V^2 \) applies.

The relevant values of design wind pressure are given in Table 2. It should be noted that the correct assessment of ground roughness category is important because it makes an appreciable difference to the design pressure. When in doubt, a higher category should be used. In unusual circumstances, when walls are constructed on very exposed hill slopes and crests where acceleration of wind is known to occur, or in valleys shaped to produce funnelling effects, the values of pressure given in Table 2, should be increased by 20%. The existence of these unusual local effects should be checked with the nearest meteorological office.

**Table 2  Design wind pressure for walls of any length not exceeding 3 m in height* (above ground level)**

<table>
<thead>
<tr>
<th>Basic Wind speed ( V ) m/sec</th>
<th>Ground roughness Category 3 Country with many windbreaks, small towns, outskirts of large cities ( p ) N/m²</th>
<th>Category 2 Open country with scattered windbreaks ( p ) N/m²</th>
<th>Category 1 Open country with no obstructions ( p ) N/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>38</td>
<td>459</td>
<td>572</td>
<td>775</td>
</tr>
<tr>
<td>40</td>
<td>509</td>
<td>634</td>
<td>859</td>
</tr>
<tr>
<td>42</td>
<td>561</td>
<td>699</td>
<td>947</td>
</tr>
<tr>
<td>44</td>
<td>616</td>
<td>767</td>
<td>1040</td>
</tr>
<tr>
<td>46</td>
<td>673</td>
<td>838</td>
<td>1136</td>
</tr>
<tr>
<td>48</td>
<td>733</td>
<td>912</td>
<td>1237</td>
</tr>
<tr>
<td>50</td>
<td>795</td>
<td>990</td>
<td>1343</td>
</tr>
<tr>
<td>52</td>
<td>860</td>
<td>1071</td>
<td>1452</td>
</tr>
<tr>
<td>54</td>
<td>927</td>
<td>1155</td>
<td>1566</td>
</tr>
<tr>
<td>56</td>
<td>997</td>
<td>1242</td>
<td>1684</td>
</tr>
</tbody>
</table>

*For walls between 3 & 4 m in height the values of the design wind pressure given in Table 1 should be increased by 9%, and for walls between 4 & 5 m the values in Table 2 should be increased by 20%.
9, 10 & 11 Satisfactorily designed wall, with adequate movement joints, suitable overhang and drip on tile-creased capping (10), and two-layer slate dpc (11).

TYPE OF WALL
The ideal characteristics of free-standing walls are:
(1) Strength and durability
(2) Economy
(3) Pleasing appearance
(4) Simplicity

The wall configurations shown in Tables 3 and 4 conform to these criteria. Traditionally, the type of wall frequently used by builders and architects comprises a 103* mm leaf with 215 mm square piers at 2 m spacing. Such a wall is at risk, even in a sheltered environment, and its strength is virtually the same as that of a 103 mm wall without piers when the direction of the wind is on the face containing the piers. Piers are effective when the wind blows on the plain face but, clearly, there is no way of ensuring this permanently.

In general, 103 mm cantilever walls – plain or with small piers – should never be more than 0.9 m high, and should be restricted to parts of the country where the design wind pressure does not exceed approximately 500 N/m².

An improved type of 103 mm wall with a small stagger is shown in Figure 4. This may be used where the design wind pressure does not exceed approximately 500 N/m², provided the wall is not higher than 1.2 m.

Simple 103 mm walls are frequently used as boundary, separation or decorative landscaping features. However, it is important to remember that timber, or even wire mesh fencing, must not be used to extend their effective height.

Where screening to ensure privacy and security is required, or where walls are used to provide a noise barrier, the useful range of heights is 1.8 m to 5 m, although the vast majority of walls will be below 3 m.

* For simplicity, half brick walls are assumed to be 103 mm.

Design of free standing walls
For walls higher than 5 m, diaphragm wall construction may be more economical. This is dealt with in other BDA publications.\(^{(a, \, b)}\).

Table 3 shows walls of the same order of strength as a 215 mm solid wall. The efficiency number is given by \(\frac{100 \times \text{section modulus} \text{ m}^3}{\text{cross-sectional area} \text{ m}^2}\) and is intended to indicate the relation between flexural strength and the volume of materials used. The higher the number, the more efficient the configuration.

Table 4 shows walls of the same order of strength as a 328 mm solid wall which can be used for a height of about 3 m. Above this height consideration should be given to the use of reinforced masonry, which is likely to be more economical.

However, if an unreinforced wall is preferred, the configuration shown in Figure 5, which has an average \(Z\) value of \(3.0 \times 10^{-4}\) m\(^3\) per m run, is suitable for heights 3.5 to 4.5 m.

![Figure 5](image)

It is not recommended that walls above 3 m high should be constructed without advice from an engineer or other appropriately qualified person. Many factors vitally affecting the strength may be quite inadvertently overlooked by the architect or the contractor, and the consequences of a failure of a high wall could be very serious to life and property. When dealing with high, expensive walls, the selection of a suitable configuration should be the subject of a close consultation between the architect, the quantity surveyor, and the engineer to arrive at an optimum solution. In the area covered by the GLC By-Laws, any free-standing walls in excess of 1.83 m in height (from ground level) must be approved by the District Surveyor.

<table>
<thead>
<tr>
<th>Table 3</th>
<th>Walls which can be used for a height of up to 2.25 m (depending on location and subject to design procedure)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of wall</td>
<td>Average cross-sectional area in m(^2) per m run</td>
</tr>
<tr>
<td>------------</td>
<td>-----------------------------------------------</td>
</tr>
<tr>
<td>1 Solid 215 mm, either bonded or collar jointed.</td>
<td>0.215</td>
</tr>
<tr>
<td>2 Staggered – GLC pattern</td>
<td>0.129</td>
</tr>
</tbody>
</table>
| 3 With piers  
Piers need not be symmetrically disposed in relation to wall,  
\(Z\) and \(I\) values are based on piers alone. | 0.184 | \(0.92 \times 10^{-4}\) | \(2.53 \times 10^{-4}\) | 4.9 |

\(a\) For wall type 2, the values for \(Z\) & \(I\) are calculated on the basis of a restricted flange width in accordance with BS 5628: Part 1\(^{10}\): clause 36.4.3.
<table>
<thead>
<tr>
<th>Type of wall</th>
<th>Average cross-sectional area in m² per m run</th>
<th>Average section property per m run of wall*</th>
<th>Efficiency rating the higher the better</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 Solid 328 mm</td>
<td>0.328</td>
<td>$1.79 \times 10^{-2}$ $2.94 \times 10^{-2}$</td>
<td>5.5</td>
</tr>
<tr>
<td>5 Staggered – GLC pattern</td>
<td>0.143</td>
<td>$1.33 \times 10^{-2}$ $2.93 \times 10^{-2}$</td>
<td>9.3</td>
</tr>
<tr>
<td></td>
<td>*See wall type 6 below.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6 Staggered – GLC pattern</td>
<td>0.156</td>
<td>$1.93 \times 10^{-2}$ $5.33 \times 10^{-3}$</td>
<td>12.3</td>
</tr>
<tr>
<td></td>
<td>*For walls type 5 &amp; 6, the values for Z &amp; I are calculated on the basis of a restricted flange width in accordance with BS 5628: Part 1(10): clause 36.4.3.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7 With piers</td>
<td>0.206</td>
<td>$1.34 \times 10^{-3}$ $4.47 \times 10^{-3}$</td>
<td>6.5</td>
</tr>
<tr>
<td>Piers need not be symmetrically disposed in relation to wall. Z and I values are based on piers alone.</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

12 A very old and tall wall at BSI Conference Centre.
Empirical selection procedure

Whilst this guide deals in full with the theoretical aspects of the subject, and thus will allow professional engineers to use its recommendations for other wall configurations, it is recognised that architects, surveyors and builders may wish to use it without having to produce lengthy calculations. To this end, the guide provides tables and diagrams which will allow a suitable wall to be selected by adopting the procedure illustrated in the following example.

EXAMPLE

Design a brick wall in the suburbs of Northampton, 2.2 m high above ground level.

Procedure

1. Locate the site of the proposed wall on the map, Figure 3, and obtain the appropriate basic wind speed, V (wind contour).
2. From Table 2, select the ground roughness category which best fits the environment of the wall, and determine the design wind pressure from the appropriate basic wind speed (wind contour).
3. Determine the effective height of wall required (see Figure 6).
4. From the graphs in Figure 7, or Table 6, select type of wall for the design wind pressure and the effective height.

5. From Tables 3 & 4, select the type of wall you prefer, bearing in mind the efficiency rating.
6. From trial holes on the site, determine the type of soil and, from Table 9, obtain the value of the allowable net bearing pressure.
7. From Table 10, obtain details of foundation corresponding to a wall type 7 bearing on loose sand.

Example

Northampton, from Figure 3, V = 42 m/sec.

From Table 2, outskirts of cities, ground roughness Category 3.
Design wind pressure, p = 561 N/m².

2.2 m wall is required. Therefore, with 0.2 m of wall below ground (see note (a) below), the effective height is 2.4 m.

Locate point on Figure 7 corresponding to wind pressure 560 N/m² and effective height 2.4 m curves to the right of this point indicate satisfactory wall types. Or, from Table 6, select nearest greater height (2.5 m). All wall types capable of resisting pressures greater than 560 N/m² are suitable. Therefore, walls type 4, 5, 6 & 7 (see Table 4) are all satisfactory.

Wall type 7 is selected. Note that, in this example, only walls shown in Table 4 are suitable.

Trial holes indicate loose sand, therefore net bearing pressure 100 kN/m².

From Table 10, relevant foundation dimensions for wall type 7, and net bearing pressure of 100 kN/m² are:

\[ a = 0.5 \text{ m}, \ b = 0.5 \text{ m}, \ c = 0.8 \text{ m}. \]

Note the following:

(a) The effective height of wall is measured from the top of the foundations, thus the ‘buried’ part of the wall should be kept to a minimum and, in general, should not exceed 200 mm.

(b) The width and thickness of the foundations are the same for all heights of wall of a given type in unreinforced brickwork.

(c) In cohesive soils (clays), trees or very large shrubs should not be close to the wall. If this is impossible, a depth of embedment of foundations of at least 1.5 m is recommended.

(d) In made-up soils (see Table 9), the use of reinforced foundations strips is recommended. These will have to be designed by a competent person.

METHODS OF DESIGN

When considering the stability of free-standing walls, the designer has three possible methods of approach:
(a) Flexural strength
(b) Notional rotation about one of the faces at the base, section virtually fully cracked
(c) Section under compression throughout

These methods are illustrated in relation to a solid wall, but are equally applicable to staggered walls and walls with piers.

(a) Design based on flexural strength (see Figure 6)
The flexural strengths of a wall is assessed in accordance with the masonry code, BS 5628(101) (see Table 15). In the guide, for simplicity, the characteristic flexural strength of masonry is taken as $f_{kx} = 0.3 \text{ N/mm}^2$. This represents the minimum value which applies to all clay and calcium silicate bricks laid in a 1:1:6 mortar, or stronger, see Table 15 (page 33).

In order to derive a simple relation between $H$ (effective height) and $p$ (design pressure), it is assumed that the critical section occurs at foundation level. Whilst, in some instances, the wall may have significant lateral support from any well-compacted soil over the base, such resistance is not always present, and quite difficult to quantify. For economical design, it is best to keep the depth of wall in the ground to a minimum. Planting of grass or flowers will require a depth of soil over the foundations of some 150 mm but, in any case, the soil cover should not exceed 200 mm.

**Design parameters and procedure**
The self weight of a masonry wall assists in its stability.

Weight of masonry, $w_m = 21000 \text{ N/m}^2$ is assumed. Where low density bricks are used, the weight of masonry may be as low as 18000 N/m$^2$. However, this will have insignificant effect on the relation between $p$ and $H$. It can be shown that for each 1000 N/m$^2$ reduction in the weight of masonry, the height of the wall, as obtained from Table 5, will be reduced by 1%.

- $w_m$, weight of masonry, $= 21 \text{ kN/m}^2$
- $\gamma_f$ for dead load $= 0.9$
- $f_{kx} = 0.3 \text{ N/mm}^2 = 300000 \text{ N/m}^2$
- $\gamma_m = 3.1$ (it is assumed that bricks used in construction are usually subject to special manufacturing control)

Whilst the factor of 3.1 is considered appropriate to the design of free-standing walls, it is recognised that some designers or regulatory authorities may prefer to use $\gamma_m = 3.5$. Accordingly, values and formulae are given in respect of both coefficients.

$Z = 0.77 \times 10^{-2} \text{ m}^3$ per metre run (for wall type 1 in Table 3).

Moments at base, $M = \frac{pH^2}{2}$ per m run, where $p$ is the design wind pressure in N/m$^2$, from Table 2, and $H$ is the height of the wall above the base in m.

$$\frac{1}{\gamma_m} f_{kx} = \frac{M}{Z} - w_m H \gamma_f$$

*Design of free standing walls*
The quadratic equation can be solved with respect to $H$ for different values of the design wind pressure, $p$, obtained from Table 2. Similar quadratic equations can be derived for the walls shown in Table 4 from the general expression, substituting appropriate values of $Z$.

### Table 5  Formulae for different types of wall

<table>
<thead>
<tr>
<th>Type of wall (see tables 3 &amp; 4)</th>
<th>Quadratic equation applicable $\gamma_m = 3.1$</th>
<th>Quadratic equation applicable $\gamma_m = 3.5$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1, 2, 3</td>
<td>$pH^2 - 293H - 1490 = 0$</td>
<td>$pH^2 - 293H - 1320 = 0$</td>
</tr>
<tr>
<td>5, 7</td>
<td>$pH^2 - 503H - 2580 = 0$</td>
<td>$pH^2 - 503H - 2280 = 0$</td>
</tr>
<tr>
<td>4, 6</td>
<td>$pH^2 - 677H - 3470 = 0$</td>
<td>$pH^2 - 677H - 3070 = 0$</td>
</tr>
</tbody>
</table>

The solutions to these equations are plotted in Figure 7 for values of the design wind pressure between 400 and 1500 N/m$^2$. Example: a wall is required near Lincoln – basic wind speed 44 m/s – in the open country with scattered windbreaks. From Table 2, the design wind pressure is 767 N/m$^2$, and from the graph in Figure 7 we read off the limiting heights of different types of wall for $\gamma_m = 3.1$:

- 1.6 m for types 1, 2 & 3
- 2.2 m for types 5 & 7
- 2.6 m for types 4 & 6

**Design based on notional rotation about one of the faces at base level**

**Design parameters and procedure:**

- Design wind pressure $p$ N/m$^2$ (values from Table 2)
- Weight of masonry, $w_m = 21$ kN/m$^2$
- $\gamma_i$ for dead load = 0.9

Assume width of stress block at point $X = \frac{1}{10}$, see Figure 8.

(This assumption is valid for all strengths of brickwork)

Moment at base, $M = \frac{pH^2}{2}$ Nm
The wall is assumed to be rotating about the centre of a very narrow stress block at X, with a tensile crack extending over almost the full width of the wall. In this condition the use of $\gamma_m$ is inappropriate and a general factor of safety $\gamma_G = 1.2$ is introduced which, together with partial factors for live and dead loads, give a global factor of safety $\gamma_{GL}$ related to the stability limit state:

$$\gamma_{GL} = \gamma_G \frac{1}{\gamma_f(\text{dead})} \times \gamma_f(\text{wind})$$

$$= 1.2 \times \frac{1}{0.9} \times 1.2 = 1.6$$

Consider rotation of the wall about the mid-point of the stress block at X, then, overturning moment x general factor = righting moment:

$$\frac{pH^2}{2} \times \gamma_G = 0.45t \times H \times t \times w_m \times \gamma_r$$

where the distance between the centre of the stress block and the centre of the wall is:

$$t - \frac{t}{20} = 0.45t$$

$$0.6pH = 0.45t^2 \times 21000 \times 0.9$$

Therefore,

$$H = 14200 \frac{t^2}{p} \text{ where } H \text{ and } t \text{ are in metres and } p \text{ in N/m}^2.$$ 

**EXAMPLE**

**Consider 328 mm wall (type 4), from the expression above:**

$$H = \frac{14200 \times 0.328^2}{p} = \frac{1530}{p}$$

<table>
<thead>
<tr>
<th>p in N/m²</th>
<th>600</th>
<th>700</th>
<th>800</th>
<th>900</th>
</tr>
</thead>
<tbody>
<tr>
<td>H in m</td>
<td>2.55</td>
<td>2.19</td>
<td>1.91</td>
<td>1.70</td>
</tr>
</tbody>
</table>

These results are more conservative than those obtainable by the application of method (a) based on the flexural strength. However, in some cases where the flexural tension cannot be relied upon, this approach will have to be adopted.

The method of notional rotation can also be applied to staggered walls types 2, 5 & 6, but these configurations are very much less efficient (being considerably lighter) under no-flexural-tension conditions. This is particularly true in the case of wall type 2, but when the stagger is greater a quite reasonable moment of resistance is available. It can be shown that, in the case of wall type 6, the relation between wind pressure and height is $H = \frac{1275}{p}$ giving results some 45% lower than those based on method (a) – flexural strength. In cases where flexural tension cannot be relied upon, this approach will be useful. It is also reassuring to know that, if a wall loses its flexural strength due to deterioration or impact, it will still retain over 50% of its original moment of resistance.
(c) Design based on section under compression throughout (no tension method)
In this approach, it is assumed that flexural tension is not allowed. Thus, the compression due to self weight, reduced by \( \gamma_r \), must equal the flexural tension due to the design wind load. Consider a 1 m run of wall H m high:

\[
H \times \bar{w}_m \times \gamma_r = \frac{pH^2}{2} \times \frac{6}{t^2} \\
H = 6300 \frac{t^2}{p}
\]

By comparing this with design method (a) or even (b), it is clear that extremely conservative results would be obtained. For example, if the wall is 215 mm and the design wind pressure 420 N/m², then \( H = 0.7 \) m.

This design approach should never be used because it leads to over-conservative results. It is mentioned here because some designers have been known, quite wrongly, to attempt to apply it to masonry walls, and because it may be legitimately used in the design of dry masonry walls.

RECOMMENDED DESIGN
Comparison of the three methods of design, clearly indicate that an economical solution should be based on method (a) – flexural strength. Walls must, therefore, be built to ensure tensile resistance, see Table 1 and section on mortars.

The three sets of quadratic equations given in Table 5 can be solved for a given magnitude of \( p \) (from Table 2) thus obtaining the appropriate value of height; but a simpler way is to substitute values of H at 0.25 m intervals and obtain the corresponding values of \( p \). In general, \( p = \frac{bH^2 + c}{H^2} \) where \( b \) and \( c \) are the coefficients of the appropriate quadratic equation from Table 5.

For example, if a wall type 4 or 6 is chosen and a height, \( H \), of 3 m is required, the allowable design wind pressure is given by:

\[
p = \frac{677 \times 3 + 3473}{9} = 612 \text{ N/m}^2, \text{ when } \gamma_m = 3.1
\]

Table 6 shows the compilation of these results for different types of walls. The required values can be obtained by interpolation, or from the graphs in Figure 7.

<table>
<thead>
<tr>
<th>Hm</th>
<th>Maximum allowable design wind pressure, ( p ), in N/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Walls type 1, 2, 3</td>
</tr>
<tr>
<td></td>
<td>( \gamma_m = 3.1 )</td>
</tr>
<tr>
<td>3.25</td>
<td>399</td>
</tr>
<tr>
<td>3.00</td>
<td>454</td>
</tr>
<tr>
<td>2.75</td>
<td>524</td>
</tr>
<tr>
<td>2.50</td>
<td>614</td>
</tr>
<tr>
<td>2.25</td>
<td>425</td>
</tr>
<tr>
<td>2.00</td>
<td>454</td>
</tr>
<tr>
<td>2.00</td>
<td>519</td>
</tr>
<tr>
<td>1.75</td>
<td>654</td>
</tr>
<tr>
<td>1.50</td>
<td>858</td>
</tr>
<tr>
<td>1.25</td>
<td>1188</td>
</tr>
</tbody>
</table>

EXAMPLE: THE USE OF TABLE 6
A 2.4 m wall is required near Newcastle, in the open country with scattered wind breaks. What type of wall would be selected?
Basic wind speed is 46 m/sec and Category 2 applies.
From Table 2, the design wind pressure is 838 N/m². Clearly, wall types 4 or 6 are suitable. Interpolation of figures from Table 6 shows that a 2.4 m wall type 4 or 6 will have a limiting design wind pressure of 890 N/m² if \( \gamma_m = 3.1 \) is chosen.

Flexural strength of 103 mm wall between piers
In the case of walls type 2, 3, 5, 6, & 7, the strength of a 103 mm wall spanning continuously between piers must be checked. The flexural stress is parallel to the bed joint (bending about the vertical axis), therefore from Table 15 (page 33) \( f_{\text{ex}} \) has a minimum value of 0.9 N/mm² (900 kN/m²) for clay and calcium silicate bricks laid in a 1:1:6 mortar.
Assume maximum span moment, \( M = \frac{pL^2}{10} \), where from Tables 3 & 4 span \( L = 1.8 \) m.

\[
M = \frac{p \times 1.8^2}{10} = 0.324p
\]

\[
Z = \frac{0.103^2 \times 1}{6} = 1.77 \times 10^{-3} \text{ m}^3 \text{ per metre}
\]

\[
\frac{1}{\gamma_m} \times f_{kx} = \frac{M}{Z}
\]

\[
\frac{1}{3.1} \times 0.9 \times 10^4 = \frac{0.324p}{1.77 \times 10^{-3}}
\]

\[
p = \frac{1.77 \times 0.9 \times 10^3}{3.1 \times 0.324} = 1586 \text{ N/m}^2, \text{ say 1600 N/m}^2
\]

Thus the wall is safe in all cases when \( p \leq 1600 \text{ N/m}^2 \).

Shear strength of piers
Consider wall type 6, which gives the lowest shear resistance (see Table 4).
When \( p = 2000 \) N/m²
Shear force on pier \( = 1.8 \times 2000 \times (1.5 - 0.2) = 4680 \) N (see Table 6)
where \( 1.5 - 0.2 \) is the height of wall above ground.

Average shear stress \( = \frac{4680}{552 \times 215} = 0.039 \text{ N/mm}^2 \)

which is well below the minimum allowable value of \( \frac{0.35}{2.5} = 0.14 \text{ N/mm}^2 \)

where 0.35 N/mm² is the characteristic shear strength of masonry laid in 1:1:6 mortar and 2.5 is \( \gamma_m \), the partial safety factor in shear.

End Condition
Walls type 1 and 4 can be stopped at any point, and walls type 3 and 7 will clearly terminate at a pier. But, in the case of walls type 2, 5 & 6 which derive their strength from the \( Z \) configuration, the end pier has to be considerably enlarged. In some instances, a gate one metre wide may be attached to the pier and thus the total length of the wind face supported by the pier is \( \frac{1.8}{2} + 0.5 = 1.4 \) m.

The required section modulus of the end pier, \( Z_e = \frac{1.4}{1.8} \times \text{section modulus of 1.8 m run of staggered wall} \).

In the case of wall type 2, for example, where the section modulus of 1.8 m run = \( 0.79 \times 10^{-2} \times 1.8 = 1.42 \times 10^{-2} \) m²:

\[
Z_e = \frac{1.4}{1.8} \times 1.42 \times 10^{-2} = 1.1 \times 10^{-2} \text{ m}^2
\]

For a pier 328 mm deep,

\[
\frac{0.328^2 \times B}{6} = 1.1 \times 10^{-2}, \text{ see Figure 9}
\]

\[
B = \frac{1.1 \times 6}{3.28^2} = 0.61 \text{ m.}
\]

Or, if the pier is made 440 mm deep:

\[
B = \frac{1.1 \times 6}{4.4^2} = 0.34 \text{ m (0.328 will do), see Table 7.}
\]
End conditions for other staggered walls are shown in Table 7. In all cases a metre wide gate was assumed attached to the end pier.

Table 7

<table>
<thead>
<tr>
<th>Wall type</th>
<th>Section modulus of end pier required in m²</th>
<th>End condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>$1.1 \times 10^{-2}$</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>$1.87 \times 10^{-3}$</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>$2.7 \times 10^{-3}$</td>
<td></td>
</tr>
</tbody>
</table>

Design of movement joints
As has already been said, masonry materials are subject to dimensional changes for a variety of causes, the most important of which relate to variations in moisture content and temperature.

The effect of moisture expansion of clay bricks will be reduced by using mature bricks. Clay brick masonry is hardly affected by drying shrinkage. It also has a low coefficient of thermal expansion.

To accommodate movement in free-standing walls of clay bricks, it is recommended that, in the case of staggered brick walls types 2, 5 & 6, 10 mm open joints should be provided in the middle of every sixth bay (see Figure 10), and for straight walls types 1 & 4, such joints should occur at about 10 m centres.

Where staggered walls are used, the bay containing the expansion joint should be reduced to 1.4 m. This is because the section modulus is 12% less due to the reduction in the length of the effective flange, one end of which is now free.

In the case of walls with piers, types 3 & 7, the bay containing the movement joint may be of standard length, ie, 1.8 m.

Where calcium silicate bricks are specified, movement joints at closer centres are recommended to avoid unsightly cracking. A gap of between 5 & 10 mm in every fourth bay – ie, at approximately 7 m centres – should be adequate for this purpose.

It may be advisable to increase the frequency of movement joints in cappings and copings.
Table 8 gives the order of free movement that might be expected in a 10 m length of wall.

<table>
<thead>
<tr>
<th>Cause of movement</th>
<th>Movement (mm) in a 10 m length of wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture expansion</td>
<td>+1.5 to 5.5 (clay)</td>
</tr>
<tr>
<td>Thermal expansion</td>
<td>2.5 (clay)</td>
</tr>
<tr>
<td>or contraction</td>
<td>4.5 (calcium silicate)</td>
</tr>
<tr>
<td>Drying shrinkage</td>
<td>-2.5 (calcium silicate)</td>
</tr>
</tbody>
</table>

Where the provision of a movement joint cuts the leaf between piers, its strength when acting as a cantilever should be checked. Figure 11 (a) shows such a movement joint in a wall type 7. Alternatively, in walls type 3 & 7 only, a movement joint may be incorporated at a pier as shown in Figure 11 (b).

To check the strength of cantilever in Fig 11 (a): \[ M = \frac{0.79^2 p}{2} = 0.31 p \text{ N/m} \] This is, in fact, a smaller moment than in the case of a continuous span (see page 21) and the cantilever is clearly adequate for the design wind pressure, \( p \), of up to 1600 N/m².

Movement joints may be left open or, where privacy is required, pointed with acrylic mastic of suitable colour. Another useful detail, shown in Fig 12, incorporates a folded neoprene baffle lightly tacked to the masonry by a suitable mastic such as acrylic.

For neat appearance, and to facilitate sealing, movement joints should be formed using a sliding board of appropriate thickness. For reinforced brickwork, movement joints may be located and constructed as recommended for unreinforced brickwork.

**Design of foundations**

Suitable foundations for walls up to 3 m high may, in general, be selected on the basis of Tables 9 & 10, together with information from a trial hole inspection carried out by a competent person. It is strongly recommended, however, that for higher walls, advice should be obtained from an engineer. In certain cases, the engineer may recommend a full ground investigation to establish the soil properties, and to confirm the absence of faults (eg, layers of organic soil or swallow holes) lower down and the stability of sloping ground.

At the time of writing this guide, the limit state approach to foundation design has not been introduced. Thus, calculations will be based on the following parameters:

(i) Wind pressure related to height and type of wall, from Fig. 8 or Table 6, divided by 1.2

(ii) Actual weight of masonry, concrete and soil

(iii) Allowable net bearing pressures from Table 9, increased by 25% for wind and by an appropriate amount for depth effect (equal to density of soil \( \times \) depth of embedment).

**EXAMPLE**

Design suitable foundations for a wall 2.5 m high, type 4, Figure 13, bearing in:

(a) soft clay

(b) loose sand

**Design data:**

- Density of masonry = 21 kN/m²
- Density of concrete = 23 kN/m²
- Density of soil = 17 kN/m²
Wind pressure for foundation design \( = \frac{826}{1.2} = 690 \text{ N/m}^2 \) (see Table 6)

Assume that, in this case, the suitable formation level is 0.7 m below the surface.

Allowable bearing pressure (soft clay)
\[ = 50 \times 1.25 + 17 \times 0.7 = 75 \text{ kN/m}^2 \]

Allowable bearing pressure (loose sand)
\[ = 100 \times 1.25 + 17 \times 0.7 = 137 \text{ kN/m}^2 \]

(a) Consider 1 m run of wall and assume 0.75 m width of foundations.

Weight of stem \( = 2.5 \times 0.328 \times 21 = 17.2 \text{ kN} \)

Weight of base \( = 0.75 \times 0.5 \times 23 = 8.6 \text{ kN} \)

Weight of soil \( = 0.75 \times 0.2 \times 17 = 2.5 \text{ kN} \)

Total \( = 28.3 \text{ kN} \)

Wind moment at formation level
\[ = 2.5 \times 690 \times (1.25 + 0.5) = 3020 \text{ Nm} \]

\[ e = \frac{3020}{28300} = 0.107 \text{ m} \]

Ground pressure
\[ = \frac{28.3 \pm 3.02 \times 6}{0.75 \pm 0.75^2} = 38 \pm 32 \]

\[ = 70 \text{ or } 6 \text{ kN/m}^2 \) (see Figure 13)

Foundation is suitable for soft clay.

*Table 9 Allowable net bearing pressure and other recommendations*

<table>
<thead>
<tr>
<th>Group</th>
<th>Type</th>
<th>Minimum allowable NET bearing pressure kN/m²</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cohesive soils</td>
<td>Stiff clay</td>
<td>150</td>
<td>Bottoms of trenches must be protected.</td>
</tr>
<tr>
<td></td>
<td>Firm clay</td>
<td>100</td>
<td>Concreting or blinding should be carried out within 2 or 3 days after excavation</td>
</tr>
<tr>
<td></td>
<td>Soft clay</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>Non cohesive soils</td>
<td>Loose gravel or loose sand and gravel</td>
<td>150 to 200</td>
<td>Water level at least 0.5 m below bottom of foundation</td>
</tr>
<tr>
<td></td>
<td>Loose sand</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>Rocks</td>
<td>Soft chalk (putty chalk)</td>
<td>100 to 150</td>
<td>Bottoms of trenches must be protected by early concreting or blinding to prevent formation of slurry</td>
</tr>
<tr>
<td></td>
<td>Other rocks</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>Made up soils</td>
<td>Mature (undisturbed for several years) mixed fill of cohesive and granular soils</td>
<td>50</td>
<td>Local soft spots, pockets of organic soil etc should be removed and back filled in lean concrete. Strips should preferably be reinforced</td>
</tr>
<tr>
<td>Made up soils</td>
<td>Newly placed mixed fill</td>
<td>25</td>
<td>Suitable for foundations of walls not exceeding 2 m in height if fill is mechanically consolidated in layers of 200 mm thickness. Foundation strips must be reinforced. Foundations may not always be entirely trouble free</td>
</tr>
<tr>
<td>Organic soils</td>
<td>Peat and organic soils</td>
<td></td>
<td>Not suitable for foundations but if the layer of organic soil or peat is at least 2 m below the underside of foundations, reinforced strip footing designed to 25 kN/m² may be installed in the overlying suitable soil. However, some risk of settlement must be accepted</td>
</tr>
</tbody>
</table>

*For many soils higher values of the allowable net bearing pressure are appropriate. However, practical considerations preclude the reduction of foundation dimensions below values given in Table 10.*
(b) Now consider foundations in loose sand and assume a width of 0.5 m. Total weight is now 24.6 kN because the strip is narrower. Wind moment is unchanged at 3020 N/m

\[
e = \frac{3020}{24600} = 0.123 \text{ m, which is outside the middle third of a 0.5 m strip}
\]

Ground pressure \(= \frac{0.5 \times 0.38}{0.5 \times 0.38} = 130 \text{ kN/m}^2\) (see Figure 14)

Foundation is suitable for loose sand.

Table 10  Recommended foundations

<table>
<thead>
<tr>
<th>Type of wall</th>
<th>Foundations</th>
<th>Max. ht. of wall measured from top of base (m)</th>
<th>Width of base (m)</th>
<th>Allowable NET bearing pressure from Table 9</th>
<th>Minimum thickness (m)</th>
<th>Minimum depth of embedment (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>2.25</td>
<td>0.5</td>
<td>0.5</td>
<td>0.3</td>
<td>0.5</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>2.25</td>
<td>0.5</td>
<td>0.5</td>
<td>0.3</td>
<td>0.5</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>2.25</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>3.25</td>
<td>0.8</td>
<td>0.6</td>
<td>0.5</td>
<td>0.5 or 0.75*</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>3</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
<td>0.5 or 0.75*</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>3.25</td>
<td>0.7</td>
<td>0.7</td>
<td>0.7</td>
<td>0.5 or 0.75*</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>3</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5 or 0.75*</td>
</tr>
</tbody>
</table>

* A minimum depth of embedment of 0.75 m is recommended for walls exceeding 2.5 m in height in cohesive soils.

Design of free standing walls
METHODS OF DESIGN
Two methods of design of reinforced brickwork are currently in use:
(1) Permissible stress method, as given in CP111, Part 2, 1970 (amended June 1971)\(^{11}\)
(2) Limit state method, for which there is as yet no British Standard Code of Practice, but which is
covered by SP 91\(^ {113}\).

PERMISSIBLE STRESS DESIGN METHOD
Figure 15 shows a reinforced brickwork section in bending.

Where:
\( d_{ef} \) - effective depth
\( z \) - lever arm
\( b \) - width of section
\( n \) - compression stress block coefficient
\( a \) - lever arm coefficient
\( P_{mb} \) - permissible bending stress in masonry
\( P_{st} \) - permissible tensile stress in reinforcement:
140 N/mm\(^2\) for mild steel
210 N/mm\(^2\) for high yield steel

Assume the stress ratio \( \frac{P_{st}}{P_{mb}} = r \), and the modular ratio \( \frac{E_s}{E_b} = m \), then from similar triangles the following
relationships are obtained:
\[
\begin{align*}
    n &= \frac{1}{1 + \frac{r}{m}} \\
    a &= 1 - \frac{n}{3} \\
    a &= \frac{2m + 3r}{3m + 3r}
\end{align*}
\]

Moments of resistance of the section can now be derived.
\[
M_e = C \times z = \frac{P_{mb}}{2} na bd_{ef} = Qbd_{ef}^2 \] is the moment of resistance based on masonry in flexure, and
\[
M_r = T \times z = A_{st}P_{st} \times ad_{ef} \] is the moment of resistance based on tension in reinforcement.

In practice, the moment of resistance will be governed by the strength of masonry in bending, and

consequently for a particular masonry the required effective depth is given by
\[
d_{ef} = \sqrt{\frac{M}{bQ}} \] where \( M \) is
the applied moment and Q is given in Table 11 or in Fig 16.

The area of reinforcement is \( A_{st} = \frac{M}{P_{st\text{ad} ef}} \)

Design parameters for the working stress method are given in Table 11.

**Table 11**

<table>
<thead>
<tr>
<th>Crushing strength of bricks in N/mm²</th>
<th>CP 111, Table 3a</th>
<th>10.5</th>
<th>20.5</th>
<th>27.5</th>
<th>34.5</th>
<th>52</th>
<th>69</th>
</tr>
</thead>
<tbody>
<tr>
<td>mortar</td>
<td>1:1:3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( p_{mb} ) N/mm²</td>
<td>1.4</td>
<td>2.2</td>
<td>2.73</td>
<td>3.33</td>
<td>4.67</td>
<td>14.67</td>
<td>14.67</td>
</tr>
<tr>
<td>( p_{st} ) (HY) N/mm²</td>
<td>210.0</td>
<td>210.0</td>
<td>210.0</td>
<td>210.0</td>
<td>210.0</td>
<td>210.0</td>
<td>210.0</td>
</tr>
<tr>
<td>( m ) (table 6a of the amendment)</td>
<td>( \frac{E_s}{E_n} )</td>
<td>33.0</td>
<td>27.0</td>
<td>24.0</td>
<td>21.0</td>
<td>15.0</td>
<td>12.0</td>
</tr>
<tr>
<td>( r ) ( \frac{D_{st}}{P_{mb}} )</td>
<td>150.0</td>
<td>95.0</td>
<td>76.9</td>
<td>63.1</td>
<td>45.0</td>
<td>45.0</td>
<td>45.0</td>
</tr>
<tr>
<td>( r ) ( \frac{m}{n} )</td>
<td>4.55</td>
<td>3.52</td>
<td>3.20</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
</tr>
<tr>
<td>( n )</td>
<td>0.18</td>
<td>0.22</td>
<td>0.24</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
</tr>
<tr>
<td>( a ), lever arm factor</td>
<td>0.940</td>
<td>0.927</td>
<td>0.920</td>
<td>0.917</td>
<td>0.917</td>
<td>0.917</td>
<td>0.930</td>
</tr>
<tr>
<td>( Q ) N/mm² (max.)</td>
<td>0.118</td>
<td>0.224</td>
<td>0.301</td>
<td>0.382</td>
<td>0.535</td>
<td>0.456</td>
<td>0.456</td>
</tr>
</tbody>
</table>

**Mortars**

Cement mortars should be specified in proportions by the volume of the dry constituents, as given below:

- **Grade (i)**
  - Cement
  - Hydrated lime
  - Sand
  - 1
  - \( \frac{1}{4} \)
  - \( \frac{3}{4} \)

- **Grade (ii)**
  - Cement
  - Hydrated lime
  - Sand
  - 1
  - \( \frac{1}{2} \)
  - \( \frac{4}{2} \)

Grade (i) is preferred.

**Grouts**

Grouts should be one of the following mixes by volume of dry materials:

- **Cement**
  - Lime (optional)
  - Sand
  - 10 mm aggregate

- **Mix (a)**
  - 1
  - \( \frac{1}{10} \)
  - \( \frac{1}{4} \)
  - 3

- **Mix (b)**
  - 1
  - \( \frac{1}{10} \)
  - \( \frac{1}{4} \)
  - 3

Mix (a) must be liquid and should be used for grouting internal joints and cavities less than 50 mm wide.

*Design of free standing walls*
Mix (b) should have a slump between 75 mm and 175 mm and should be used to fill grout spaces which are 50–100 mm wide.

No additives should be used in mortar, grout or concrete, but carefully controlled amounts of colouring agents may be used.

**Minimum area of reinforcement**

Although the Code (CP111, Part 2) does not specify the minimum percentage of reinforcement in flexural members, the author considers that 0.1% of the gross cross-sectional area of the member should be provided.

**Reinforced Quetta bond, pocket & grouted cavity walls**

*EXAMPLE, PERMISSIBLE STRESS DESIGN METHOD*

Design a free standing wall 3 m high, using three different types of construction: Quetta bond, pocket and grouted cavity. Basic wind speed is 50 m/s, ground roughness Category 1. Bricks with a crushing strength of 34.5 N/mm² in 1:1:3 mortar are specified in all cases.

\[ p_{mb} = 3.33 \text{ N/mm}^2 \text{ (see Table 11).} \]

\[ m = 21 \text{ from Table 6a, CP111, Part 2 (Amendment)} \]

\[ p_{st} = 210 \text{ N/mm}^2 \text{ for high tensile steel (CP111, Part 2)} \]

\[ r = \frac{210}{3.33} = 63 \]

\[ n = \frac{1}{1 + \frac{21}{63}} = 0.25 \]

\[ a = 0.917 \]

\[ Q = \frac{3.33}{2} \times 0.917 \times 0.25 = 0.38 \text{ N/mm}^2 \]

\[ M_r = 0.38bd_{ef}^2 \]

Alternatively, \( Q \) may be obtained from the graph in Fig. 16, or Table 11.

From Table 2, for basic wind speed 50 m/s, ground roughness Category 1, we have:

Design wind pressure for limit state design \( = 1343 \text{ N/m}^2 \)

Design pressure for permissible stress design \( = \frac{1343}{1.2} = 1120 \text{ N/m}^2 \)

Design bending moment, \( M = \frac{pH^2}{2} \)

\[ M = \frac{1120 \times 3^2}{2} = 5040 \text{ Nm} = 5.04 \times 10^6 \text{ Nmm} \]

The minimum required effective depth, \( d_{ef} \) can now be calculated:

\[ d_{ef} = \frac{\sqrt{5.04 \times 10^6}}{\sqrt{0.38 \times 0.917 \times 164 \times 210}} = 115\text{mm} \]

Therefore, the wall section in Figure 17 is satisfactory and, because the actual \( d_{ef} \) is 164 mm, a brick of lower crushing strength would have been adequate.

![Wall Section Diagram](image)

\[ A_{st} = \frac{5.04 \times 10^4}{0.917 \times 164 \times 210} = 160 \text{ mm}^2, \text{ or 0.1\% of gross area = 328 mm}^2 \]

Provide 12 mm HY bars in alternate pockets, ie, at approximately 340 mm c/c.

\( A_{st} \text{ provided} = 332 \text{ mm}^2 \)
Pocket wall (Figs. 18 & 19)
Use 215 mm overall thickness collar jointed wall with 200 × 110 mm staggered pockets at about 2 m c/c on both sides of the wall. Bending moment is as in the previous example.

\[
Q = \frac{M}{b d_{ct}^2} = \frac{5.04 \times 10^6}{10^3 \times 160^2} = 0.2, \text{ therefore proposed strength of masonry is ample.}
\]

\[
z = 0.917 \times 160 = 147 \text{ mm}
\]

\[
A_{st} = \frac{5.04 \times 10^6 \times 2}{0.917 \times 160 \times 210} = 327 \text{ mm}^2 \text{ per 2 m run}
\]

or, 0.1% of gross area = 2000 × 215 × 10⁻³ = 430 mm².

Provide two 16 mm HY bars in each pocket, \(A_{st} = 402 \text{ mm}^2\).

Percentage of reinforcement = \(\frac{402 \times 100}{1000 \times 215} = 0.18\)

Shear stress = \(1120 \times 3\)

\[= \frac{10^3 \times 160 \times 0.917}{1000 \times 215} = 0.023 \text{ N/mm}^2, \text{ is satisfactory.}
\]

A suitable arrangement of staggered pockets at 2 m spacing, on alternate sides of the wall, is shown in Fig. 18.

The appearance of the wall will be considerably improved if the pockets are faced with brickwork, attached to the concrete by suitable ties, as shown in Fig. 19.

Grouted cavity wall (Fig. 20)
The wall consists of two half-brick leaves with a 100 mm grouted cavity.

Moment of resistance, \(M_r = 0.38 \times 1000 \times 153 = 8.9 \times 10^6 \text{ Nmm. But, since the applied moment is} \)

\(5.04 \times 10^6 \text{ Nmm (see Quetta bond example) the wall is adequate.}

\[
A_{st} = \frac{5.04 \times 10^6}{0.917 \times 153 \times 210} = 171 \text{ mm}^2
\]

or, 0.1% of gross area = 306 \(\times 10^3 \times 10^{-3} = 306 \text{ mm}^2\).

Provide 12 mm HY bars at 350 mm c/c, \(A_{st} = 323 \text{ mm}^2\).

The secondary reinforcement should be at least 0.05% of the gross cross-sectional area of the wall.

Area required = 306 \(\times 10^3 \times 5 \times 10^{-4} = 153 \text{ mm. Provide 10 mm bars at 500 mm c/c.}

LIMIT STATE DESIGN METHOD
Principles
This design approach is based on BCRA Special Publication 91(12). The design moments of resistance are given below. The lesser of the two values should be taken as the resistance of the section in bending.

\[
M_d = \frac{0.3b d_{ef} \gamma_{ms}^2}{\gamma_{mm}^2} \text{ based on the strength of brickwork in bending}
\]

or,

\[
M_d = \frac{A_s f_{ez}}{\gamma_{ms}^2} \text{ based on the strength of reinforcement in tension}
\]

where

\[
z = d_{ef} \left(1 - \frac{0.53 A_s \times f_y}{b d_{ef} \times f_{ez} \times \gamma_{ms} \times 1.5 f_k} \right) \leq 0.95 d_{ef}
\]

The expression in brackets is the lever arm coefficient, \(a\), which has a range of 0.8 ≤ \(a\) ≤ 0.95.

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\[ A_s = \frac{M \gamma_{ms}}{2f_y} \]

\( M \) = applied design moment
\( M_d \) = design moment of resistance
\( b \) = width of section
\( d_{ef} \) = effective depth of section
\( f'_t \) = characteristic tensile strength of the reinforcement, given in Table 14
\( f'_k \) = characteristic compressive strength of brickwork, given in Table 12
\( a \) = lever arm coefficient (see also page 34)
\( z \) = lever arm
\( \gamma_{nm} \) = partial safety factor for brickwork strength, given in Table 13
\( \gamma_{ms} \) = partial safety factor for reinforcement strength which should be taken as 1.15

### Table 12: Characteristic compressive strength of brickwork, \( f'_k \), normal to bed joints (N/mm\(^2\)) for the design of reinforced masonry to limit state

<table>
<thead>
<tr>
<th>Mortar grade and mix</th>
<th>Compressive strength of brick (N/mm(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>15</td>
</tr>
<tr>
<td>(i) 1:1:3</td>
<td>6.0</td>
</tr>
<tr>
<td>(ii) 1:1:4(\frac{1}{2})</td>
<td>5.3</td>
</tr>
</tbody>
</table>

### Table 13: Partial safety factors \( (\gamma_{nm})^* \) for strength of brickwork in compression and flexure applicable to reinforced and unreinforced masonry designed to limit state

<table>
<thead>
<tr>
<th>Category of construction control</th>
<th>Normal</th>
<th>Special</th>
</tr>
</thead>
<tbody>
<tr>
<td>Category of manufacturing control of structural units</td>
<td>Normal</td>
<td>3.5</td>
</tr>
<tr>
<td>Special</td>
<td>3.1</td>
<td>2.5</td>
</tr>
</tbody>
</table>

*For reinforced masonry this factor is designated \( \gamma_{nm} \)

### Table 14: Characteristic strength of reinforcement for limit state design

<table>
<thead>
<tr>
<th>Designation</th>
<th>Nominal sizes (mm)</th>
<th>Characteristic strength ( f'_k ) (N/mm(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hot-rolled mild steel (BS 4449)</td>
<td>All sizes</td>
<td>250</td>
</tr>
<tr>
<td>Hot-rolled high yield (BS 4449)</td>
<td>All sizes</td>
<td>410</td>
</tr>
<tr>
<td>Cold-worked high yield (BS 4461)</td>
<td>Up to and incl. 16</td>
<td>460</td>
</tr>
<tr>
<td>Hard-drawn steel wire (BS 4482)</td>
<td>Over 16</td>
<td>425</td>
</tr>
<tr>
<td></td>
<td>Up to and incl. 12</td>
<td>485</td>
</tr>
</tbody>
</table>

The shear stress is given by:

\[ v = \frac{V}{bd_{ef}} \]

where
\( V \) = shear force due to design loads
\( b \) = width of section
\( d_{ef} \) = effective depth

The shear stress, \( v \), should not exceed the characteristic shear strength of brickwork \( f'_{vb} = 0.35 \text{ N/mm}^2 \) divided by the partial safety factor \( \gamma_{mv} \) which is taken as 2.5.

Shear reinforcement is most unlikely to be required, and in any case its use is not recommended in these simple structures.
Slenderness limits for reinforced cantilever walls
The ratio of span to effective depth of a cantilever wall, with up to 0.5% reinforcement based on gross cross-sectional area, should not normally exceed 18. However, where the load is transient (wind), and there is no danger of damage to finishes, the slenderness ratio limit may be increased by 30% to 23.4.

Reinforced Quetta bond wall

**EXAMPLE**

Design a reinforced free standing wall 3 m high constructed in Quetta bond, using limit state design method. Basic wind speed is 56 m/s, and ground roughness Category 1 are assumed.

Design moment \( M = \frac{pH^2}{2} = 1684 \times \frac{3^2}{2} = 7578 \text{ Nm} \). Thickness of wall in this case is governed by the slenderness limit. Minimum \( d_{ef} = \frac{3000}{23.4} = 130 \text{ mm} \). Use 328 mm wall as shown in Figure 21. The effective depth of 164 mm is satisfactory.

Assume 20 N/mm² bricks in 1:1:3 mortar, \( f_k = 7.4 \text{ N/mm}^2 = 7.4 \times 10^3 \text{ kN/m}^2 \) (see Table 12) and \( \gamma_{mm} = 3.1 \) (see Table 13).

The maximum design moment of resistance can now be calculated:

\[
M_d = \frac{0.3 \times 10^3 \times 164^2 \times 1.5 \times 7.4}{3.1 \times 10^3} = 28900 \text{ Nm},
\]

which is ample in relation to the applied moment.

Assuming MS reinforcement (\( f_y = 250 \text{ N/mm}^2 \), see Table 14), and \( z = 0.9 \times 164 \text{ mm} \):

- \( A_s = \frac{7.58 \times 10^3 \times 1.15}{250 \times 164 \times 0.9} = 236 \text{ mm}^2 \)
- or, 0.1% of gross cross-sectional area = \( 328 \times 10^3 \times 10^{-3} = 328 \text{ mm}^2 \)

Provide 12 mm MS in alternate pockets. Since the minimum area of reinforcement governs, the assumption of the approximate lever arm as 164 \( \times 0.90 \) has no practical significance.

Shear stress = \( \frac{1684 \times 3}{10^3 \times 164} = 0.03 \text{ N/mm}^2 \). This is less than the design shear strength

which is \( \frac{0.35}{2.5} = 0.14 \text{ N/mm}^2 \).

Note: The minimum percentage of reinforcement recommended by reference (12) is 0.2% of the gross cross-sectional area. The author feels that this percentage is rather high for simple boundary walls and, consequently, 0.1% has been adopted throughout these examples.

Reinforced pocket wall

**EXAMPLE**

Design a 215 mm pocket wall of maximum height using limit state approach. Basic wind speed of 56 m/s, and ground roughness Category 1 are assumed as in previous example.

Referring to Figure 22, effective depth, \( d_{ef} = 160 \text{ mm} \). Therefore maximum permitted height = 23.4 \( \times 0.16 = 3.7 \text{ m} \). Therefore a 3.7 m wall will be satisfactory.

\[
M = \frac{pH^2}{2} = 1684 \times \frac{3.7^2}{2} = 11530 \text{ Nm}.
\]
20 N/mm² bricks in 1:1:3 mortar are specified (as in the previous example), \( \gamma_{mm} \) is taken as 3.1 (see Table 13).

Maximum design moment of resistance
\[
M_d = \frac{0.3 \times 10^8 \times 160^2 \times 1.5 \times 7.4}{3.1 \times 10^3} = 27500 \text{Nm}, \text{which is ample.}
\]
\( A_s = \frac{11530 \times 10^3 \times 1.15}{460 \times 160 \times 0.90} = 200 \text{mm}^2 \)

or, 0.1\% of the gross cross-sectional area
\[= 215 \text{mm}^2 \text{(all per m run).} \]

Provide staggered pockets of 200 \times 110 mm approximately, at about 2 m c/c on each face as shown in Figure 18. Area of reinforcement in each pocket = 400 mm², provide one 16 mm and two 12 mm HY bars, \( A_s = 427 \text{mm}^2 \). The concrete pockets can be faced with brickwork, as shown in Figure 19. For 215 mm bonded walls, use 1 brick wide pockets at 9 brick centres.

**Half brick wall stiffened with reinforced piers**

**EXAMPLE**

Design a wall 4.25 m high with reinforced piers at about 2.5 m c/c. Design wind speed is 46 m/s, and ground roughness Category 2.

Design wind pressure from Table 2 is 838 N/m² but, because the wall is just 4 m high above ground level, this pressure should be increased by 9\% (see footnote to Table 2). Design wind pressures = 838 \times 1.09 = 913 N/m².

Consider 103 mm unreinforced brickwork spanning continuously between piers.

Clear span is 2.035 m, see Figure 23, and the effective span = clear span + thickness of wall = 2.14 m.

![Figure 23](image)

Take:
\[
M = \frac{WL}{10} = \frac{913 \times 2.14}{10} = 418 \text{Nm}
\]
\[
Z = \frac{103^2 \times 1000}{6} = 1.77 \times 10^6 \text{mm}^3 \text{for 1 m width of wall}
\]

\( \gamma_{mm} \) is taken as 3.1 (see Table 13)

Flexural tension is parallel to the bed joint and therefore the two last columns in Table 15 apply.

\[
f_{ks} = \frac{418 \times 10^3 \times 3.1}{1.77 \times 10^6} = 0.73 \text{N/mm}^2
\]

From Table 15 (page 33), bricks with an absorption of over 12\% in at least grade (iii) mortar (1:1:6) will suffice. However, for reinforced piers, grade (i) mortar (1:1:3) is preferred. Thus, both the wall and the piers will be constructed in this mortar using bricks of 20 N/mm² strength.

Slenderness ratio of pier
\[
= \frac{4250}{310} = 14
\]

Load on pier = 913 \times 2.5 = 2290 N/m

\[
\frac{M}{2} = \frac{2290 \times 4.25^2}{2} = 20682 \text{Nm}
\]

\[
M_d = \frac{0.3 \times 440 \times 310^2 \times 1.5 \times 7.4}{3.1 \times 10^3} = 45420 \text{Nm, which is ample.}
\]

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Assume a lever arm coefficient of 0.9.

\[ A_s = \frac{M_{Y \text{ms}}}{f_r \times z} = \frac{2.07 \times 10^7 \times 1.15}{460 \times 0.9 \times 310} = 185 \text{ mm}^2 \]

Provide four 12 mm HY bars, percentage of reinforcement = \( \frac{1.5 \times 100 = 0.23}{440 \times 310} \)

The lever arm coefficient can now be verified using \( A_s = 185 \text{ mm}^2 \) and \( f_r = 460 \text{ N/mm}^2 \):

\[ a = \left( 1 - \frac{0.53 \times 185 \times 460 \times 3.1}{440 \times 310 \times 1.15 \times 1.5 \times 7.4} \right) = 1 - 0.08 = 0.92. \]

Therefore the lever arm coefficient of 0.9 is satisfactory.

Check shear stress, \( \nu = \frac{2290 \times 4.25}{440 \times 310} = 0.07 \text{ N/mm}^2 \). Section safe since the maximum design shear strength = \( \frac{0.35}{2.5} = 0.14 \text{ N/mm}^2 \).

<table>
<thead>
<tr>
<th>Table 15</th>
<th>Characteristic flexural (tensile) strength of masonry, ( f_{ks} \text{ N/mm}^2 ) for the design of unreinforced masonry to limit state</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plane of failure</td>
<td>Plane of failure</td>
</tr>
<tr>
<td>parallel to bed joints</td>
<td>perpendicular to bed joints</td>
</tr>
<tr>
<td>Mortar designation</td>
<td>Mortar proportion by volume</td>
</tr>
<tr>
<td>(i)</td>
<td>(ii) and (iii)</td>
</tr>
<tr>
<td>1:3:4</td>
<td>1:1:3</td>
</tr>
<tr>
<td>1:1:6</td>
<td>1:1:6</td>
</tr>
<tr>
<td>Mortar proportion by volume</td>
<td>Clay bricks having a water absorption</td>
</tr>
<tr>
<td></td>
<td>less than 7%</td>
</tr>
<tr>
<td></td>
<td>between 7% and 12%</td>
</tr>
<tr>
<td></td>
<td>over 12%</td>
</tr>
<tr>
<td></td>
<td>0.3</td>
</tr>
<tr>
<td>Calcium silicate bricks</td>
<td></td>
</tr>
</tbody>
</table>

### Simplified method for deriving lever arm coefficient in limit state design

The expression for the lever arm coefficient in limit state design:

\[ a = \left( 1 - \frac{0.53A_s f_r}{bd_r Y_{\text{mm}}} \right) \leq 0.95 \]

is rather cumbersome, and cannot be used directly to calculate the area of reinforcement from:

\[ A_s = \frac{M_{Y \text{ms}}}{f_r a d_{ef}} \]

It can be shown that the lever arm coefficient lies between 0.8 and 0.95 hence, in practice, the best procedure is to assume that \( a = 0.9 \) and calculate \( A_s \) on the basis of \( z = 0.9d_{ef} \). The value of \( A_s \) thus obtained can be substituted into the formula to verify the assumption.

A better method, however, is to express the moment of resistance in the form

\[ M_d = Q bd_{ef}^2 \text{ where it can be shown that } Q = 2.834 \left( 1 - \frac{f_k}{Y_{\text{mm}}} \right) \]

The units of \( Q \) are dictated by \( f_k \) and must be compatible with those of \( b \) and \( d_{ef} \).

The variables in this expression have the following limits:

- lever arm coefficient, \( a \) : 0.8 – 0.95
- characteristic compressive strength, \( f_k \) : 5.3 – 24 N/mm²
- partial safety factor, \( Y_{\text{mm}} \) : 2.5 – 3.5
- \( f_k \) : 1.5 – 9.6 N/mm²
- \( Y_{\text{mm}} \) : 0.2 – 4.35 N/mm²

For any particular value of \( \frac{f_k}{Y_{\text{mm}}} \) the maximum value of \( Q \) is obtained when \( a = 0.8 \) and equals \( 0.45 \frac{f_k}{Y_{\text{mm}}} \).

*The derivation of this expression is beyond the scope of this design guide.*
Design procedure

(1) Obtain \( Q = \frac{\text{design moment}}{bd_{ef}^2} \)

(2) Select \( f_k \) and \( \gamma_{mm} \) so that \( Q \leq 0.45 \frac{f_k}{\gamma_{mm}} \)

(3) Substitute the obtained value of \( Q \) from (1) and the selected value of \( \frac{f_k}{\gamma_{mm}} \) from (2) into

\[
Q = 2.834a(1-a)\frac{f_k}{\gamma_{mm}}
\]

and hence by solving a simple quadratic equation find \( a \).

The process is further simplified by plotting and tabulating all the variables in their appropriate limits as shown in Fig 24 and Table 16. This allows the value of \( a \) to be obtained directly.

Table 16  Values of \( Q \) (units as for \( f_k \))

<table>
<thead>
<tr>
<th>( a )</th>
<th>( \frac{f_k}{\gamma_{mm}} )</th>
<th>1.00</th>
<th>2.00</th>
<th>3.00</th>
<th>4.00</th>
<th>5.00</th>
<th>6.00</th>
<th>7.00</th>
<th>8.00</th>
<th>9.00</th>
<th>10.00</th>
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<tr>
<td>0.95</td>
<td>0.135</td>
<td>0.269</td>
<td>0.404</td>
<td>0.538</td>
<td>0.673</td>
<td>0.808</td>
<td>0.942</td>
<td>1.077</td>
<td>1.212</td>
<td>1.346</td>
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<tr>
<td>0.92</td>
<td>0.209</td>
<td>0.417</td>
<td>0.626</td>
<td>0.834</td>
<td>1.043</td>
<td>1.251</td>
<td>1.460</td>
<td>1.669</td>
<td>1.877</td>
<td>2.086</td>
<td></td>
</tr>
<tr>
<td>0.89</td>
<td>0.277</td>
<td>0.555</td>
<td>0.832</td>
<td>1.110</td>
<td>1.387</td>
<td>1.665</td>
<td>1.942</td>
<td>2.220</td>
<td>2.497</td>
<td>2.774</td>
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<tr>
<td>0.86</td>
<td>0.341</td>
<td>0.682</td>
<td>1.024</td>
<td>1.365</td>
<td>1.706</td>
<td>2.047</td>
<td>2.388</td>
<td>2.730</td>
<td>3.071</td>
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<td></td>
</tr>
<tr>
<td>0.83</td>
<td>0.400</td>
<td>0.800</td>
<td>1.200</td>
<td>1.600</td>
<td>1.999</td>
<td>2.399</td>
<td>2.799</td>
<td>3.199</td>
<td>3.599</td>
<td>3.999</td>
<td></td>
</tr>
<tr>
<td>0.80</td>
<td>0.453</td>
<td>0.907</td>
<td>1.360</td>
<td>1.814</td>
<td>2.267</td>
<td>2.721</td>
<td>3.174</td>
<td>3.628</td>
<td>4.081</td>
<td>4.534</td>
<td></td>
</tr>
</tbody>
</table>
REFERENCES
