DESIGN GUIDE NO. 2.

THE DESIGN OF BRICKWORK RETAINING WALLS

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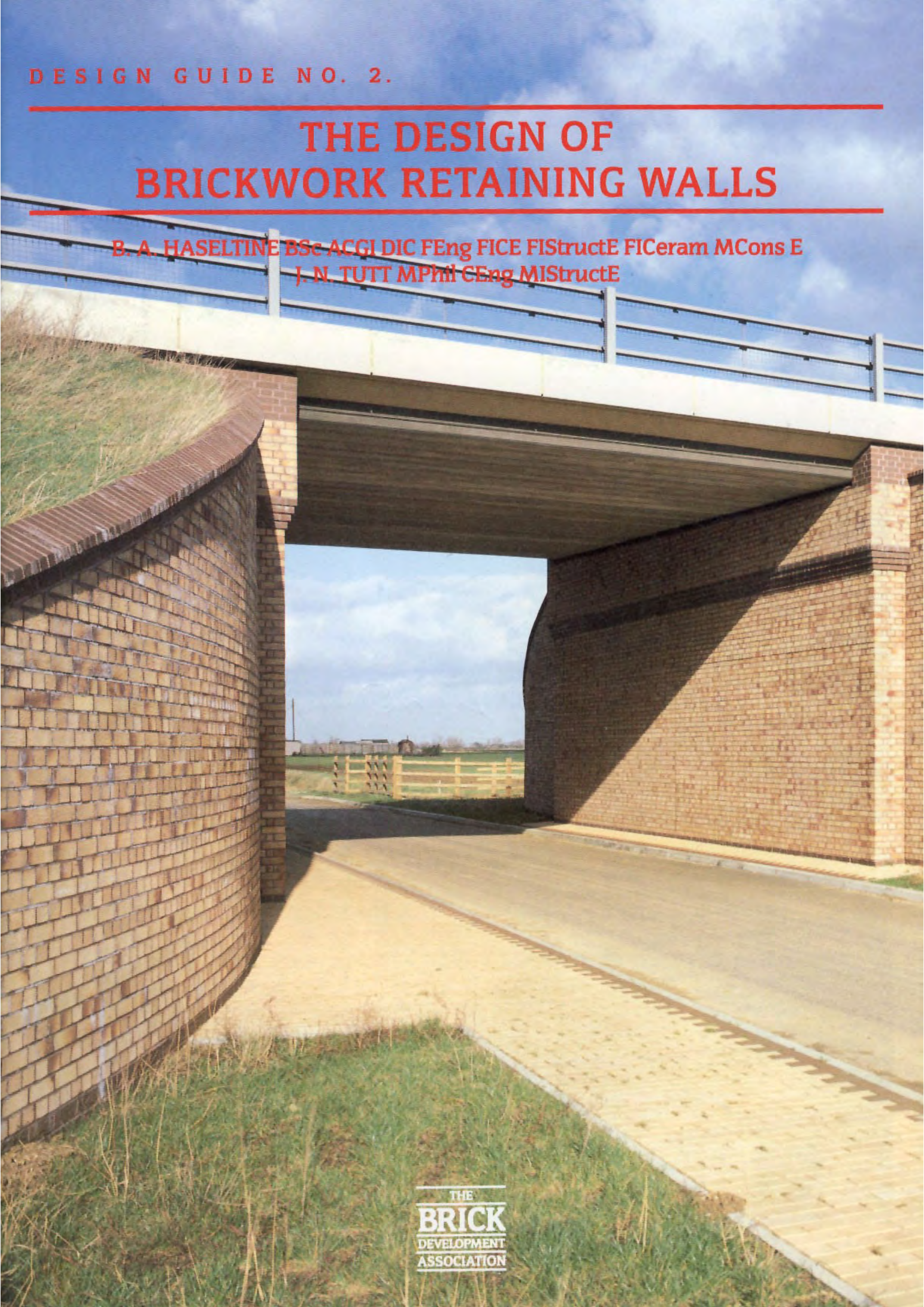

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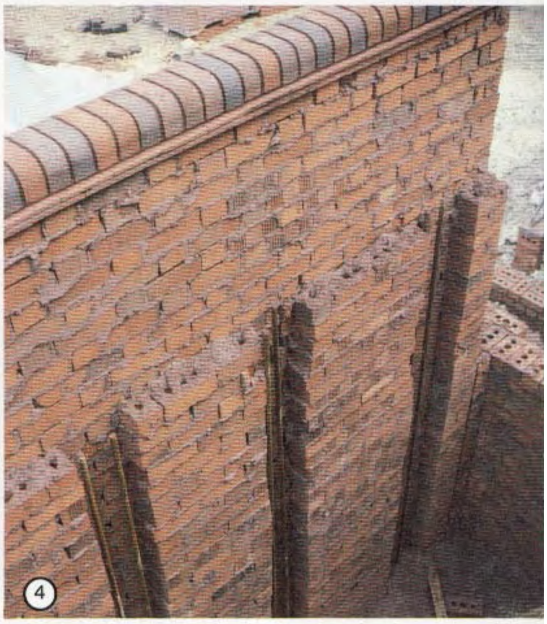
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Accommodation works for a road widening scheme in a General Improvement Area involved the provision of new car parking bays to serve existing houses, located at a level of some 4m or 5m above the carriageway.

Retaining walls approximately 3m high were required and pocket reinforced brick retaining walls offered the following advantages:

- Simplicity of design
- Ease of construction
- Economy of construction and material
- Versatility of size and form
- Attractive appearance without secondary decoration or surface treatment
- Robustness

The design and supervision of construction were undertaken by W A J Sketch, B.Sc.,CEng.,FICE.,FIHT., County Engineer, Bucks County Council.

The General Improvement Area project was undertaken by Wycombe District Council.

1. General shot of Phase 1 walls with parking bays. Steps between, lead up to the houses.

2. Example of 3 car bays showing retaining wall.

3. Construction of wall 1½ bricks thick with pockets on rear face to accommodate reinforcing bars, anchored into concrete foundations at 900mm centres, (maximum) - irregularity of pockets is intentional to improve keying of infill concrete grout.

4. Completed wall - pockets cleaned out preparatory to grouting.

5. Temporary shuttering placed against pockets and strutted.

6. Shuttering struck after infill concrete has fully matured. Usually super plasticised, pea gravel, high slump infill concrete is used (BS5628: Part 2).

7. Back of wall primed with bituminous emulsion.

8. Self adhesive damp proof membrane. Note land drain at base of wall.

9. Completed retaining wall after back filling.

CONTENTS

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First published August 1977
Revised edition February 1979
Reprinted August 1981
Revised Edition September 1991

EDITOR'S NOTE

The 1991 revision of this design guide is similar to the 1981 edition. The opportunity has been taken during the revision to update the publication with respect to changes in the Building Regulations, and with respect to the publication BS5628:Part 2 in 1985. Similarly, a fuller treatment is given to soil mechanics aspects in this edition. While the answers obtained from this design guide should be broadly similar to answers obtained using the previous edition, the method of calculation presented here complies with the recommendations of BS5628:Part 2 (see Introduction for further comments).

Research has indicated that values of the characteristic flexural strength of brickwork given in Table 3 of BS5628:Part 1 may be too high for walls of 215 mm thickness or greater.

At the time of going to press, it was understood that an amendment to the Code was being considered in the light of a paper by R Lovegrove "The effect of thickness and bond pattern upon the lateral strength of brickwork", in Proc. Brit. Mas. Soc. **2**, 95, April 1988. Readers are referred to this.

High Lift Grouted Cavity Construction

Top
Grouted cavity wall ready for infiiling. Note wall ties to BS5628:Part 2, Appendix B.

Middle
Boarding guide for placing infill concrete: sheeting to avoid splashing and staining of fair faced brickwork.

Lower
Compaction of infill concrete with vibrator.

SYMBOLS

A — plan cross-section area of wall
A_s — cross-section area of reinforcement
A_{sv} — cross-sectional area of reinforcement resisting shear forces
a_1 — lever arm factor
b — length of wall under consideration, usually 1.0m (width of section)
C — total compressive force
c — cohesion
c_o — shear strength (cohesion) at zero normal load
d — effective depth: limit state design
d_{ef} — effective depth of reinforcement: permissible stress design
e — eccentricity
$F_{1,2}$ — forces due to $p_{1,2}$ etc
F_r — frictional force under base
f_b — characteristic anchorage bond strength between mortar or concrete infill and reinforcement
f_k — characteristic compressive strength of brickwork
f_{kx} — characteristic flexural strength of brickwork
f_v — characteristic shear strength of brickwork
f_y — characteristic tensile strength of reinforcement
g — acceleration due to gravity
H — height of retaining wall
H_k — overall depth of toe to base
h — depth of water table surface below top of wall
K_A — active ground pressure coefficient
K_{AC} — active ground coefficient for cohesive soils
K_p — coefficient of passive resistance of soils possessing friction
K_{pc} — coefficient of passive resistance of soils possessing cohesion
L_b — length of retaining wall base
M — applied bending moment: permissible stress design
design bending moment: limit state design
M_d — design moment of resistance - limit state design
M_r — moment of resistance - permissible stress design
m — modular ratio
n — depth of neutral axis: permissible stress design
P — total active lateral load on retaining wall
P_a — lateral load due to active earth pressure
P_p — lateral load due to passive earth pressure
P_w — lateral load due to water pressure
p — A_s/bd
$p_{1,2}$ — combined ground pressures
p_{an} — horizontal component of active earth pressure
p_b — permissible compressive strength of brickwork in flexure
p_c — permissible compressive strength of brickwork in direct compression
p_p — passive earth pressure
p_{st} — permissible tensile stress in reinforcement

p_t — permissible tensile stress in brickwork
p_v — permissible shear strength of brickwork
p_w — water pressure
Q — moment of resistance coefficient: permissible stress design
R_v — vertical component of resultant of forces on retaining walls
T — total tensile force
t — overall thickness of wall
t_f — thickness of flange in pocket-type retaining wall
V — shear force due to design loads
v — design shear stress
$W_{1,2}$ — vertical loads due to wall, base, etc
$w_{1,2}$ — pressure due to $W_{1,2}$,etc
x — height of line of action of P above bottom of wall stem
x_n — depth to neutral axis: limit state stress block
y — depth to centre of compression: limit state stress block
Z — section modulus
z — lever arm
γ_f — partial safety factor for load
γ_m — partial safety factor for strength of brickwork
γ_{mb} — partial safety factor for bond strength between mortar or concrete infill and steel
γ_{mm} — partial safety factor for compressive strength of brickwork
γ_{ms} — partial safety factor for strength of steel
γ_{mv} — partial safety factor for brickwork shear strength
ρ — density of soil
ρ_b — submerged density of soil
ρ_d — dry density of soil
ρ_s — saturated density of soil
ρ_w — density of water
ø — angle of internal friction
δ — angle of friction between retained earth and back of wall

1.0 INTRODUCTION

Brickwork has long been accepted as a building material which is highly sympathetic to, and harmonious with, our landscape. This, allied to its structural properties, makes brickwork an eminently suitable material for the construction of retaining walls, especially when its appearance is compared with that of alternative materials. Brickwork retaining walls may be constructed in plain unreinforced brickwork, when the dead weight of the retaining wall is relied upon for stability, or they may also contain reinforcement in a similar manner to reinforced concrete.

Reinforced brickwork has been widely and successfully used in the United States, India and other countries. It is now finding increasing usage in Britain, where plain brickwork has been used extensively for retaining walls of considerable size, for example, in railway cuttings.

This design guide discusses the types of brickwork retaining walls which can be used and gives information to enable them to be designed to the permissible stress methods, and to limit state methods in accordance with BS5628:Parts 1[1] and 2[2]. Example calculations are given. The costs of brickwork retaining walls, whether plain or reinforced are clearly important, and are compared, in section 5.0, with retaining walls built of other materials. The figures show that, compared with walls built in other materials of similar aesthetic quality, brickwork retaining walls are more economical.

There are, of course, other approaches to the design of brickwork retaining walls – one being the adaptation of the diaphragm wall principle. Details of this are provided in the BDA publication 'Brick diaphragm walls in tall single-storey buildings'. Alternatively, prestressed brickwork can be used to produce economical results. Guidance on this can also be found in BS5628: Part 2, but it is outside the scope of this publication.

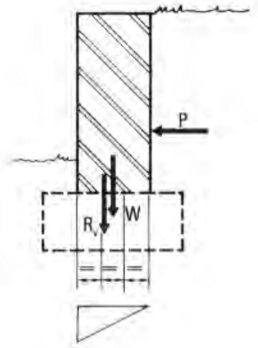
Before the publication of the first part of BS5628, in limit state terms, in 1978, the major part of the structural design of brickwork was carried out using permissible stresses in accordance with CP111[3]. Since 1978 both codes have been used in parallel, until CP111 was withdrawn by BSI in 1985. However, the new Building Regulations [4], effective from November 1985, included references in approved document for structure A1/2, to both CP111 and BS5628: Parts 1 and 3[5]. These Codes of Practice were permitted to be used to meet the requirements of Paragraphs A1 and A2 of Part A of schedule 1 to the Building Regulations 1985 for the structural work in masonry, i.e. design in accordance with these codes satisfies the Regulation. CP111 has subsequently been withdrawn by BSI.

As with the previous Building Regulations, any means of meeting the requirements of the Regulations may be adopted provided it can be shown to be adequate. Thus the use of a permissible stress approach to design is not precluded by the withdrawal of CP111, provided it can be shown to meet the requirements of the Regulations, and, perhaps of more practical importance, can be demonstrated to meet these requirements to the satisfaction of the Local Authority or Approved Inspector, as appropriate. A permissible stress approach is considered by many engineers to be particularly applicable to earth retaining wall design. It should also be noted that BS5628: Part 2 is not yet referred to in the approved document, requiring the designer using the code to satisfy the Local Authority or Approved Inspector of its suitability. Published, as it is by BSI, as Part 2 of a suite of parts of which 1 and 3 are already listed, this should be a formality.

2.0 TYPES OF BRICKWORK RETAINING WALLS

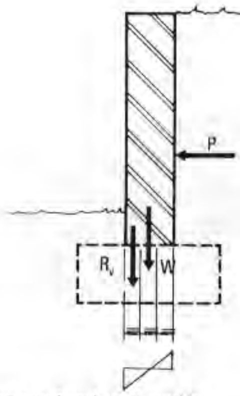
2.1 MASS BRICKWORK RETAINING WALLS

Mass retaining walls in plain brickwork rely on their dead weight to resist the overturning force arising from the retained material. They may be designed on the gravity principle, where no tension is permitted in the brickwork (Figure 1), or on the basis that small tensile stresses may be allowed to develop (Figure 2).



vertical component of resultant, R_v within middle third of wall

Figure 1. No tension



vertical component of resultant, R_v outside middle third; small tensile stress permitted

Figure 2. Permissible tension

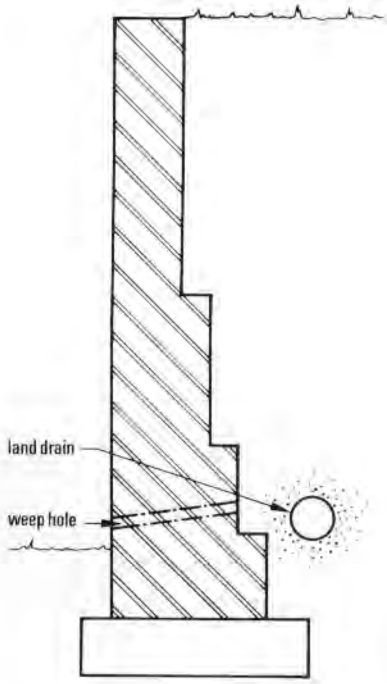


Figure 3. Stepped retaining wall

The retaining face of the wall may be stepped as shown in Figure 3 to take advantage of the reduction of lateral loading as the retained height decreases.

2.2 GROUTED CAVITY RETAINING WALLS

Grouted cavity retaining walls consist of two single skins of brickwork separated by a cavity which usually contains reinforcement. The cavity is either filled with concrete as the work proceeds, or is filled in suitable lifts when the brick skins have attained sufficient strength. Unreinforced grouted cavity walls are designed as mass walls. Reinforced grouted cavity walls are designed, in flexure, as cantilever beams.

For retained heights of over 2.0m an increase in the thickness of the outer leaf may be necessary, though if visually acceptable, it may be stepped back as the reduction in pressure with height permits.

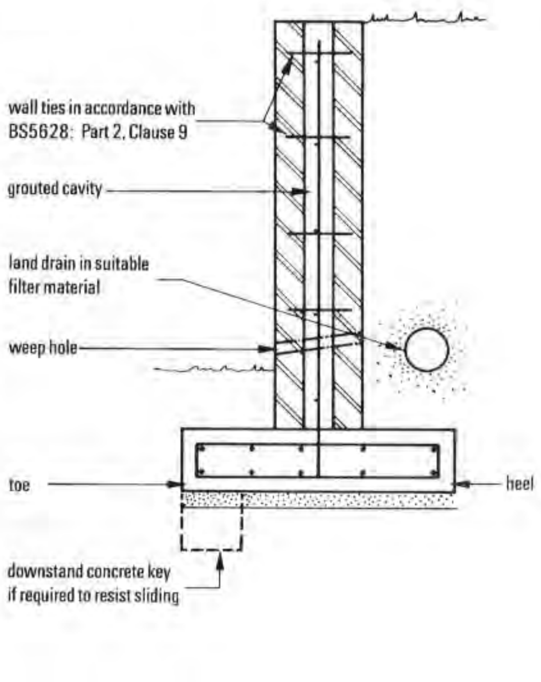


Figure 4. Grouted cavity wall

To obtain the maximum lever arm, the vertical reinforcement may be offset in the cavity, provided that it has the appropriate minimum cover given in section 4.0.

2.3 POCKET RETAINING WALLS

Pocket retaining walls contain reinforced concrete piers built into pockets in the tension face of the brickwork. The walls are designed to cantilever above their bases, as a series of flanged beams. The brickwork between pockets is considered as the compression flange. If the pockets are at greater than 1.0m centres, it will be necessary to ensure that the brickwork will span horizontally between the pockets. For a given thickness, this form of construction has the advantage of making the effective depth to the reinforcement, and, hence the strength of the wall, a maximum. For the limitations on flange width see 3.4.3.

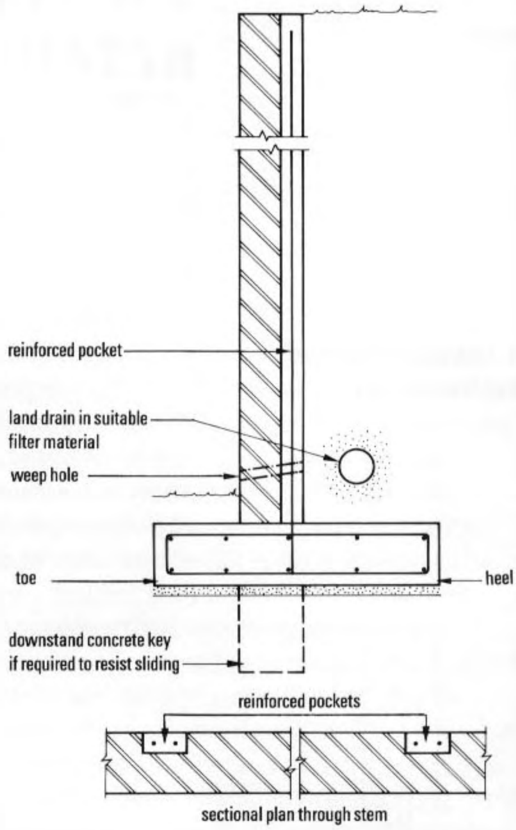


Figure 5. Pocket wall

2.4 SPECIAL BONDS

Reinforced brickwork retaining walls may be constructed using bricks laid in special bonds. The most common of these is the $1\frac{1}{2}$ brick thick Quetta bond which may be used to retain a maximum height of material of approximately 3.0m. The cavities left by the brick bonding dictate the centres of the reinforcement. This reinforcement may be offset in the cavities to make the moment of resistance of the wall a maximum. Variations on Quetta bond allow greater thickness, and hence height. Specially shaped bricks (grooved to receive reinforcement) may also be used for retaining walls.

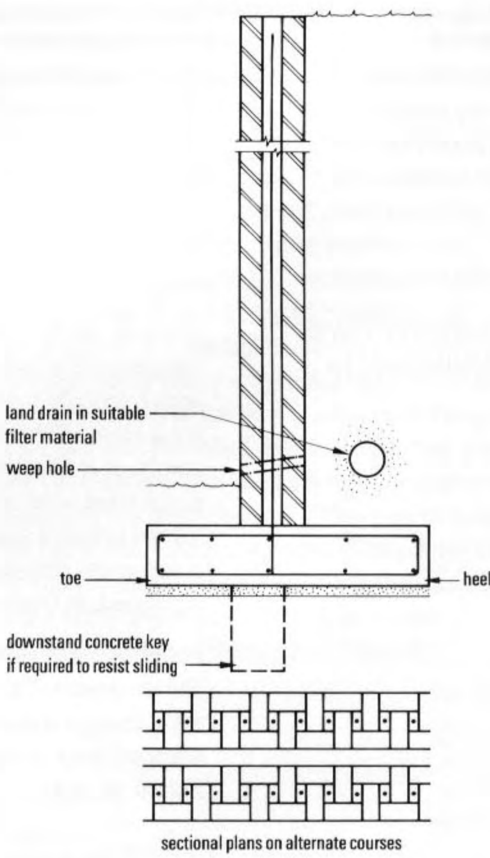


Figure 6. Quetta bond wall

3.0 DESIGN OF BRICKWORK RETAINING WALLS

3.1 DERIVATION OF LATERAL LOADS

The principal factor governing the lateral load on a retaining wall is the material to be retained. Generally the lateral load can only be estimated because of the variability of the soils retained by an earth retaining wall. In order to make the estimate as accurate as possible, the type and properties of the retained soil should be obtained from an adequate site investigation.

A perfectly rigid earth retaining wall resists a lateral load which is known as the earth pressure at rest. In practice if the top of the wall is unrestrained, i.e. the wall cantilevers vertically and is not attached to any other structure at the top, some lateral movement of the top of the wall will take place under load. This movement has the effect of reducing the lateral load on the wall from the 'earth pressure at rest' to the 'active earth pressure'.

As well as resisting the active earth pressure, retaining walls must also be designed to resist the lateral load from any water present behind the wall and from any imposed surcharge load applied to the retained earth. Where possible, the build up of water pressure behind the wall should be prevented by the provision of a drainage medium and land drain behind the wall, and weep holes through it, as shown in Figure 39.

The materials retained by brickwork earth retaining walls are generally classified as either granular (non-cohesive) materials, e.g. sand, or cohesive materials, e.g. clay. An intermediate group having some properties of granular and cohesive materials can also be identified, e.g. silts. A knowledge of soil mechanics is necessary to design retaining walls because of the fundamental differences in properties and behaviour of the materials.

Having obtained the soil properties from the site investigation, the lateral forces on a retaining wall may be assessed in accordance with Civil Engineering Code of Practice No. 2[6], or with methods given in other accepted soil mechanics literature[7]. These are generally based upon Rankine's theory or the wedge theory, which enable retaining walls with sloping faces and sloping retained surfaces to be dealt with. For simple cases where the retaining face of the wall is vertical and the surface of the retained material horizontal, calculation of the lateral load may be carried out in accordance with the information given below.

Granular (non-cohesive) materials

For dry non-cohesive retained materials without surcharge, the active earth pressure, p_{an}, can be obtained from Rankine's theory using Mohr's circle:

$$p_{an} = K_A \rho H \qquad \text{......1}$$

Where ρ is the density of the soil

H is the retained height of the soil

and K_A is the coefficient of active ground pressure.

$$= \frac{1 + \sin \varnothing}{1 - \sin \varnothing}$$

Where ø is the angle of internal friction of the soil.

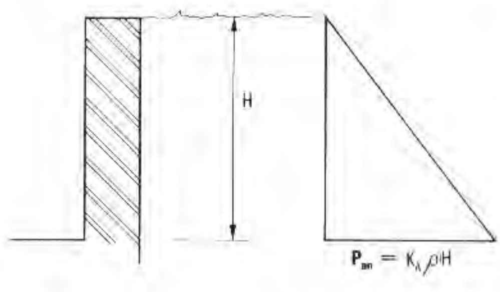


Figure 7. Lateral pressure due to granular material

TABLE 1.
Typical values of ø, ρ and ρ_b for granular (cohesionless) materials.

Material	ø	ρ* (kN/m³)	ρ_b (kN/m³)
gravel	35° – 45°	16 – 20	10 – 13
compact sand	35° – 40°	16 – 22	10 – 13
Loose sand	30° – 35°	14 – 19	10 – 13
rock filling			
granite	35° – 45°	16 – 21	10 – 13
basalt & dolorite	35° – 45°	17 – 22	11 – 16
limestones & sandstones	35° – 45°	13 – 19	6 – 13
chalk	35° – 45°	10 – 13	3 – 6
broken brick	35° – 45°	11 – 18	6 – 10
ashes	35° – 45°	7 – 10	3 – 5

* upper values for sand allow for damp sand above water table.

Typical values of ø, ρ and submerged density, ρ_b, are given in Table 1.
Note that equation 1 above neglects friction between the retained material and the back of the retaining wall. This is conservative because the friction has the effect of reducing the lateral pressure.

Values for K_A are given in Table 2.

TABLE 2: Values of K_A for cohesionless materials, vertical walls with horizontal ground.

ø	K_A
25°	0.41
30°	0.33
35°	0.27
40°	0.22
45°	0.17

Cohesive materials

Rarely will brickwork retaining walls be required to resist undisturbed cohesive materials, since working room will be required between the wall and the retained material. It is possible that the working space will be back filled with cohesive material, although the use of a granular material would be preferable for drainage purposes.

For lateral load purposes, cohesive materials and silts may be considered together in three divisions, namely:

a) non-fissured clays

b) silts and partially saturated clays

c) stiff fissured clays

The principal difference between cohesive and non-cohesive retained materials is that the former are to some degree self supporting. This is characterised by the formation of a tension crack at the top of the retained material behind the wall.

For a vertical wall with level retained material of non-fissured clay, Bell's modification of Rankine's theory gives:

$$p_{an} = \rho_s H - 2c \quad \ldots\ldots\ldots 2$$

Where ρ_s is the saturated density of the clay
c is the cohesion of the retained material

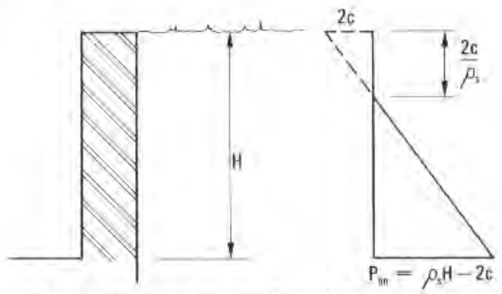


Figure 8. Lateral pressure due to non-fissured clay

It can be seen that p_{an} will be negative if $H < \frac{2c}{\rho_s}$

Thus $H = \frac{2c}{\rho_s}$ represents the depth of the tension crack

Note that equation 2 neglects adhesion between the wall and the clay and this is conservative.

Table 3 gives values of cohesion for clays of varying stiffness and also values of the saturated density of the clay, ρ_s, which should generally be used in equation 2.

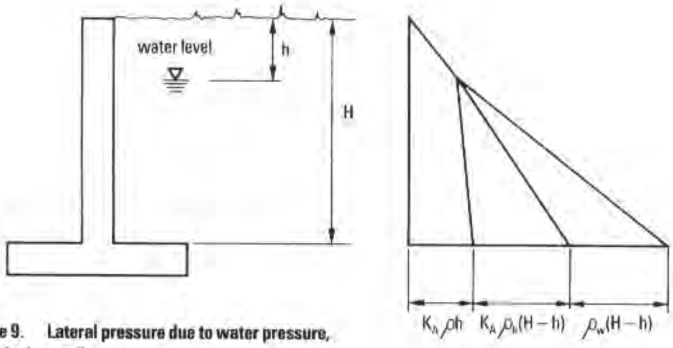


Figure 9. Lateral pressure due to water pressure, cohesionless soil

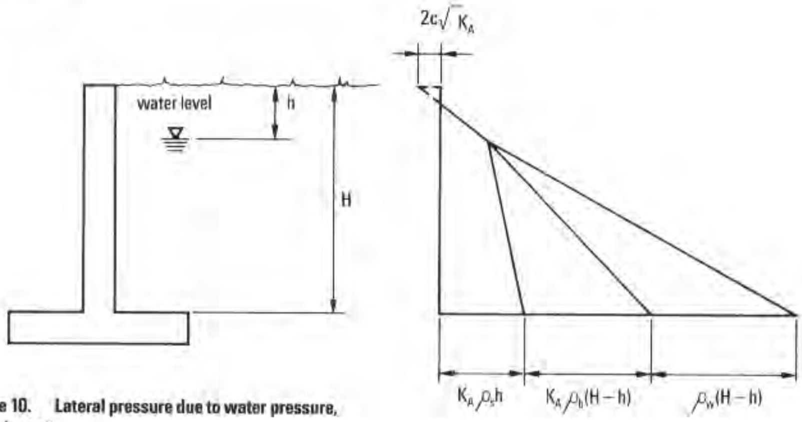


Figure 10. Lateral pressure due to water pressure, cohesive soil

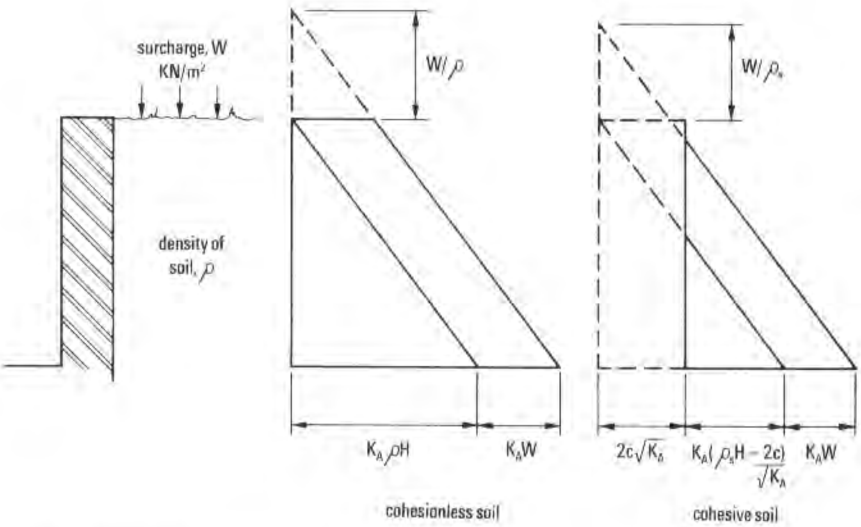


Figure 11. Effect of surcharge loading on lateral load pressure diagrams

TABLE 3 Typical values of cohesion and saturated density. Cohesive soils.

Clay type	Cohesion, $c(kN/m^2)$	saturated density ρ_s (kN/m^3)
very stiff clay	140	19 - 22
stiff clay	70 - 140	19 - 22
firm clay	35 - 70	17 - 20
soft clay	18 - 35	16 - 19
very soft clay	18	16 - 19

For soils possessing both cohesion and friction:

$p_{an} = K_A\rho H - 2c\ \sqrt{K_A}$ 3

Where drainage is limited ø is often taken as zero so that equation 3 reverts to that for non-fissured clays given above. Again adhesion to the rear of the wall is neglected, as is the angle of friction between the retained earth and the wall. Values of c and ø should, where required, be obtained from tests on these soils. Table 4 gives values of K_A for cohesive soils.

TABLE 4 Values of K_A for cohesive soils.

ø	0°	5°	10°	15°	20°	25°
K_A	1.0	0.85	0.70	0.59	048	0.40

Stiff fissured clays should be treated as non-fissured clays except that the softened value of cohesion should be used. The value to be used in a particular case is a matter of judgement; a range of values may be taken as 15 to 40 kN/m^2.

It is recommended that walls in cohesive soils are never designed for an active earth pressure of less than 4.8 times the height in metres of the retained material.

Water pressure

Where adequate drainage is provided, it may be unnecessary to make any allowance for water. However, if there is any doubt regarding the ability of the drainage to cope or where no drainage is provided, the lateral hydraulic pressure must be added to the active pressure due to submerged soil. In cohesive soils the pressure of water in the tension crack must be allowed for.

The water pressure acting normal to the wall, p_w must be added to the active earth pressure.

Thus for cohesionless soil.

$p_{an} + p_w = K_A\rho h + K_A\rho_b(H - h) + \rho_w(H - h)$

Where ρ_b is the submerged density of soil
ρ_w is the density of water
and h is the depth to the water table
Values of ρ & ρ_b are given in Table 1.

For cohesive soil, the soil above the water table will be saturated and

$$p_{an} + p_w = K_A[\rho_s h + \rho_b(H - h)] - 2c/K_A + \rho_w(H - h)$$

Where ρ_s = saturated density of soil
and ρ_b = submerged density of soil
$= \rho_s - \rho_w$
Values of c and ρ_s are given in Table 3.

Surcharge

Where the retained material is subjected to a uniformly distributed surcharge loading a further lateral pressure is applied to the rear of the wall. To calculate this pressure the surcharge is assumed to have the effect of increasing the height of the retained material by an amount equal to the intensity of the surcharge divided by the density of the retained material.

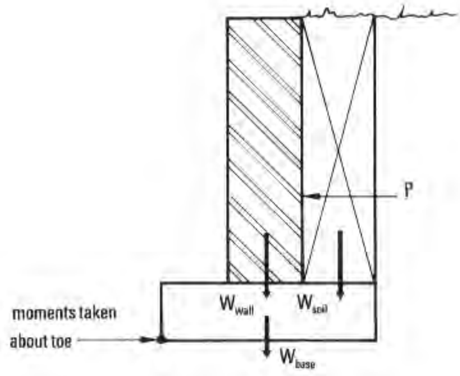
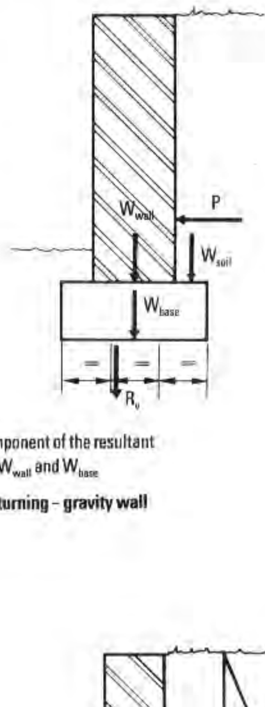


Figure 12. **Overturning**

3.2 OVERALL STABILITY

There are several potential types of failure of a retaining wall that must be avoided. Instability of the soil mass in which the wall is constructed, e.g. slip circle failure, is a soil mechanics problem which the designer must deal with by the use of standard soil mechanics procedures. Failure of the structure of the wall is prevented by good design (see section 3.3 and 3.4). There remain three modes of failure which involve the overall stability of the wall, and for which a suitable factor of safety must be provided.



R_v = vertical component of the resultant of P, W_{soil}, W_{wall} and W_{base}

Figure 13. **Overturning – gravity wall**

(a) Overturning about the toe.

To avoid the risk of the retaining wall overturning about the toe of the base, the moments due to the lateral forces causing overturning must be resisted by the righting moments due to the vertical loads of the structure and any earth carried on the heel, as shown in Figure 12, such that a factor of safety of at least 2.0 is obtained.

By ensuring that the resultant of all the forces at the level of the base is within the middle half of the base, a factor of saftey against overturning of 2.0 is obtained. Thus a gravity wall, where the resultant falls within the middle third, will inherently have an adequate factor of safety against overturning (see Figure 13)

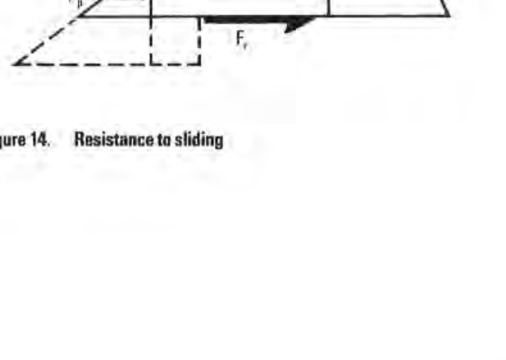


Figure 14. **Resistance to sliding**

b) Forward movement by sliding

To avoid the risk of failure of a wall by sliding, the force causing sliding, P, must be resisted by base friction on cohesionless soils, or by adhesion on cohesive soils, and the passive pressure at the toe. On cohesionless soils, if the base is cast in situ, the coefficient of friction should be taken as tan ø, where ø is the angle of internal friction of the soil; if the base is not cast in situ, the coefficient of friction may be taken as 0.35. On cohesive soil, adhesion may be taken as equal to the cohesion up to a maximum value of $40kN/m^2$. Stiff-fissured clays are subject to long-term softening and reduced values of cohesion, making allowance for this softening, should be used. The passive pressure of the earth in front of the base may be considered, providing it has not been disturbed during construction and there is no likelihood of its removal at a later date. Where these factors are insufficient to resist sliding it will be necessary to provide a key on the underside of the base to increase the passive resistance. The factor of safety against sliding should be not less than 2.0. The forces causing and resisting sliding are shown in Figure 14.

c) Shear failure of the soil below the toe of the wall

The net pressure under the toe of the retaining wall base should not exceed the bearing capacity of the soil. Values of safe bearing capacities for different soil types are quoted in CP2[6] and in other texts[7]. These values, being safe capacities, include factors of safety against shear failure of the soil. If ultimate bearing capacities are calculated, using Prandtl's or Terzaghi's methods for example, a factor of safety of at least 2 should be used in cohesionless soils to obtain the safe bearing capacity. In cohesive soils a factor of safety of at least 3 should be used to minimise the settlement under the toe of a retaining wall base.

3.3. PERMISSIBLE STRESS DESIGN

3.3.1. Mass brickwork retaining walls

a) Design of stem

Until 1978 as mentioned in Section 1.0 the structural design of brickwork was carried out exclusively in accordance with CP111:1970[3] which, although expressly excluding the design of walls subjected to lateral loading, allowed the designer, at his own discretion, to use permissible tensile stresses in bending of $0.07N/mm^2$ and $0.14N/mm^2$, depending on the direction of bending. The limit state code for the design of structural masonry, BS5628 Part 1[1], covers the design of walls subjected to lateral loading. The permissible stresses given in Table 7 have been derived from those in BS5628 Part 1 by the application of a global factor of safety of 4.5.

Mass brickwork retaining walls may be designed for there being no tension in the stem, on the basis of the middle third rule (see Figure 15) or by allowing a small tensile stress to develop (see Figure 16).

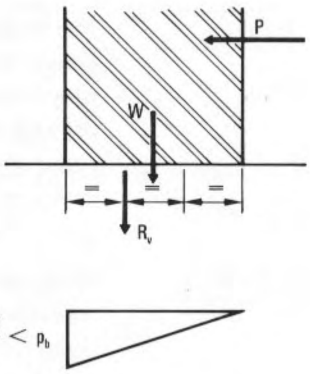


Figure 15. Stress diagram

Vertical component of resultant, R_V is within the middle third of the stem at its base, therefore, no tension occurs in the brickwork

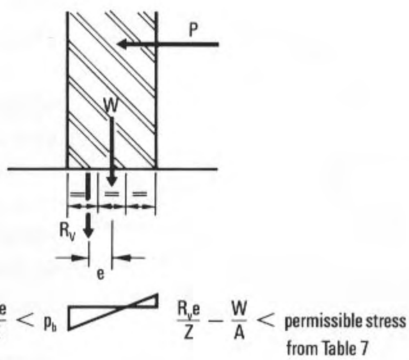


Figure 16. Stress diagram

Vertical component of resultant, R_v is outside the middle third, therefore, tension occurs

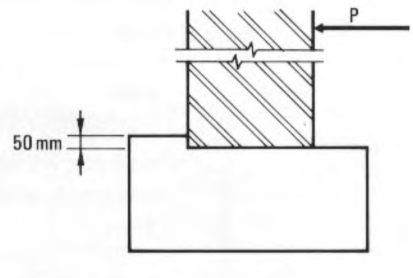


Figure 17. Stepped base to mass brickwork retaining wall

The shear stress in the brickwork due to the lateral load, P, should not exceed the permissible shear stress. The permissible shear strength given in Clause 317, of CP111 was $0.1\ N/mm^2$. The application of a global factor of safety to the characteristic shear strength of brickwork given in BS5628: Part 1 would result in a value of the same order. To avoid shear failure at the brickwork concrete interface a step may be provided as shown on Figure 17.

b) Design of base

The base of a mass brickwork retaining wall is generally unreinforced. It should, therefore, be proportioned so that the spread of load through the base does not allow tension to occur at the heel of the base or the bearing pressure under the toe to exceed the safe ground bearing capacity.

c) Buttressed walls

When vertical buttresses are introduced into a retaining wall, the panels of brickwork between them will span two ways. Analysis of these panels can be carried out in a number of ways, e.g. plate theory, yield line theory, etc. When analysing such panels, the height to length ratio of the panel - known as the aspect ratio - and the vertical to horizontal flexural strength of the brickwork - known as the orthogonal ratio - must be taken into account. Suggested permissible stresses for bending about a vertical axis are given in Table 7. For further guidance on the design of these panels, reference should be made to the BDA publication 'External walls, design for wind loads', [8]. The buttresses must be capable of withstanding the forces applied to them by the panels. Counterfort walls are not recommended as the lateral load on the panel tends to force the wall away from the counterforts.

3.3.2. Reinforced brickwork retaining walls.

a) Design of stem

The design of reinforced brickwork retaining walls to permissible stresses follows the general principles of analysis and elastic design given in CP114:1969[9]. A modular ratio approach is adopted, the values of the modular ratio and permissible stresses being given in Amendment No 1 (June 1971) to CP111. The stress block assumed is as shown on Figure 18 below.

It should be noted that, as with CP111, CP114 has been withdrawn by BSI and that the final paragraph of the Introduction also applies to CP114.

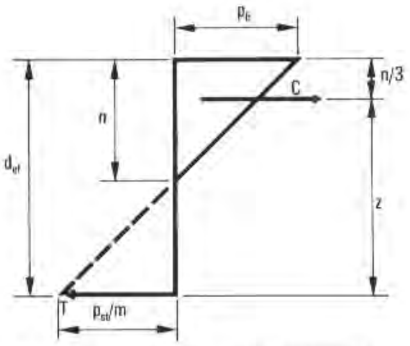


Figure 18. Stress block-modular ratio approach

The depth to the reinforcement, d_{ef}, will be known from the thickness of the wall, and from similar triangles the depth to the neutral axis, n, can be calculated. The lever arm, z, can then be found enabling the moment of resistance of the wall, M_r, to be calculated from: $M_r = Cz$.

The total compressive force, C, equals the total tensile force, T, therefore:

$M_r = Cz = Tz = A_s p_{st} z$ 4

Where C = the total compressive force
z = the lever arm
T = the total tensile force
A_s = the area of tensile reinforcement
p_{st} = the permissible tensile stress in the reinforcement.

Equating the moment of resistance to the applied moment, the required area of reinforcement, A_s, is obtained.

The shear strength of the wall should also be checked. The permissible shear stress may be taken as 0.1 N/mm^2 as given above, CP111 suggested that if this is exceeded suitably placed shear reinforcement must be provided, but this is difficult in practice. The alternative solution is to increase the stem thickness locally.

The local and average bond stresses around the reinforcement should be checked in accordance with CP114[9], using the stress given below taken from Amendment No 1 to CP111:1970[3]

Design Stresses: Brickwork, CP111:1970.
Flexural compression, p_b:4/3 x the permissible direct compressive stress (from Table 3a, CP111:1970) up to a limit of 4/3 x the permissible direct compressive stress for 52.0 N/mm^2 bricks.
Shear,p_v: 0.1N/mm^2 for brickwork with zero dead load compressive stress, varying linearly up to a maximum of 0.5 N/mm^2. These stresses apply for brickwork built with mortar not weaker then 1:1:6. According to CP111, the provision of suitably placed reinforcement permits the shear stress to be increased to a maximun of 0.5 N/mm^2 for all dead load compressive stresses. However, shear reinforcement cannot be easily provided.

Bond: between mortar and steel, 0.56 N/mm^2.

Design Stresses: Reinforcement
Tension, mild steel: 140 N/mm^2
all other steel: 0.5 x Characteristic strength up to a limit of 210 N/mm^2

TABLE 5 Moment of resistance coefficient, Q, - permissible stress design

crushing strength of bricks p_c(N/mm²)	modular ratio m	compressive strength of brickwork in bending p_b(N/mm²)	mild steel reinforcement p_{st} = 140 N/mm² Q	a_1	high yield reinforcement p_{st} = 210 N/mm² Q	a_1
10.5 - 14.0	33	1.63	0.206	0.907	0.155	0.932
14.1 - 20.5	30	2.20	0.314	0.893	0.242	0.920
20.6 - 27.5	27	2.73	0.416	0.885	0.324	0.913
27.6 - 34.5	24	3.33	0.532	0.879	0.417	0.908
34.6 - 41.5	21	3.87	0.624	0.877	0.489	0.907
41.6 - 48.5	18	4.40	0.698	0.880	0.548	0.909
48.6 - 55.0	15	4.67	0.691	0.889	0.535	0.917
over 55.0	12	4.67	0.603	0.905	0.457	0.930

Where Q = Moment of resistance coefficient
a_1 = lever arm factor

In order to simplify the design procedure, Table 5 gives moment of resistance coefficients, calculated by the modular ratio approach previously discussed. Using these coefficients the moment of resistance of a wall of particular thickness and strength of brick, may be calculated from equation 5.

Alternatively, using the applied moment, the effective depth required and therefore the minimum wall thickness, may be calculated from equation 5.

$$M_r = Qbd_{ef}^2 \qquad \ldots\ldots 5$$

The values of p given are for brickwork built with 1:¼:3 mortar.

From equation 5 the required effective depth, d_{ef}, can be found and thus the area of reinforcement from

$M = A_s\, a_1\, d_{ef}\, p_{st}$

(b) Design of Base

The base to a reinforced brickwork retaining wall will usually be constructed of reinforced concrete and can be designed to CP114 (but see 3.3.2)

The bending moments for which the base is designed are calculated from the net pressure on the underside of the base (see Figure 19).

Bending moments on base

$$M \text{ at compression face} = F_1 e_1 - \frac{w_3 \times L_1^2}{2}$$

$$M \text{ at tension face} = F_2 e_2 - \frac{(w_1 + w_3) \times L_2^2}{2}$$

where W_1 = weight of earth,
W_2 = weight of stem
W_3 = weight of base
$w_{1,2}$ etc, = pressure due to $W_{1,2}$ etc,
$p_{1,2}$ etc, = pressure from overturning analysis
$F_{1,2}$ = force due to $\frac{(p_1 + p_2)\, L_1}{2}$

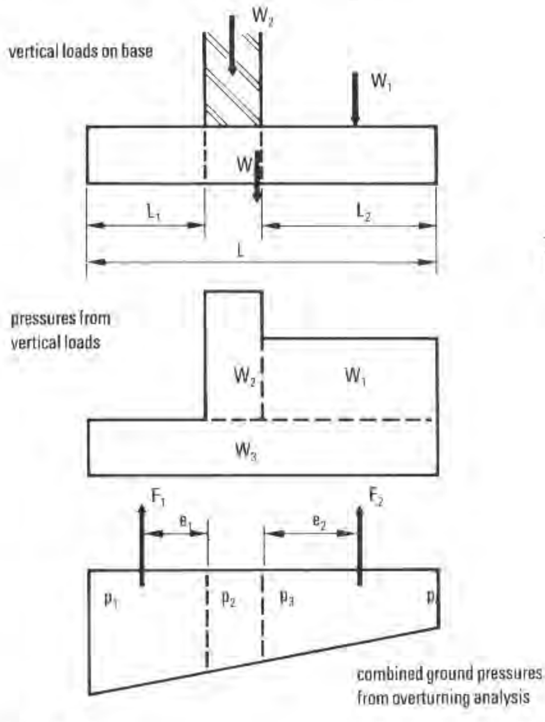


Figure 19.

$$\text{and } \frac{(p_3 + p_4)\,L_2}{2}$$

respectively

$e_{1,2}$ = distance from line of action of F_1 and F_2 respectively to face of wall.

(c) Buttressed Walls

When reinforced brickwork retaining walls are built with buttresses the panels between them will span in two directions. The bending moments for which the panels are designed should be calculated using plate, yield line, or other theory which is accepted for reinforced concrete design. The area of reinforcement required in each direction can then be calculated using the procedure given in 3.3.2(a).

In grouted cavity walls the provision of horizontal reinforcement to resist bending is easily achieved. In Quetta bond and pocket walls the horizontal reinforcement required must be positioned in the bed joints and it is therefore limited in diameter. In pocket walls, if the spacing of the reinforced concrete pockets exceeds 1.0 metre the pockets, in effect, become buttresses and the walls must be designed to span between them in bending or arching.

3.3.3. Examples

The brick dimensions used in the following examples are work sizes as defined in BS3921:1985[10]

Example 1

A mass brickwork retaining wall is to retain 1.2m of granular soil of dry density 1800 kg/m^3, and an angle of internal friction,ø, of 35°. The surface of the retained material is level and the friction between the retained earth and the back of the wall may be neglected, ie $\delta = 0$. The ground at the level of the base has a safe bearing capacity of 100 kN/m^2. Design the wall using each of the three following methods:

(i) no tension is permitted in the brickwork

(ii) a permissible tensile stress of 0.07 N/mm^2 may be used (CP111:1970, para 316).

(iii) the permissible tensile stresses from Table 7 may be used.

Figure 20 (opposite)

(i) No tension is permitted in the brickwork
Laterial earth pressure on the wall.
From Table 2, Section 3.1. the value of the coefficient of active pressure,
$K_A = 0.27$ for ø = 35°.

$$\text{Thus, } p_{an} = \frac{0.27 \times 1800 \times 9.81 \times 1.2}{1000} =$$

5.72 kN/m^2

$$P_a = p_{an} = \frac{H}{2} = 5.72 \times 0.6 = 3.43 \text{ kN/m}$$

Where p_{an} = horizontal component of active earth pressure
P_a = lateral load due to active earth pressure

Design of Stem

The overturning moment, M, =

$$P_a \frac{H}{3} = 3.43 \times 0.4 = 1.37 \text{ kNm/m.}$$

Assume a three brick wall in brickwork of density 2000kg/m^3.
Thickness = 3 x 215 + 2 x 10 = 665mm.

To ensure that no tension occurs in the brickwork, the resultant of the vertical and horizontal forces must lie within the middle third of the base of the wall, i,e., the nett eccentricity must not exceed

$$\frac{665}{6} = 110.8 \text{ mm}$$

Therefore, taking moments about A:

$$P_a \frac{H}{3} \leqslant R_v\, e \quad \text{.......... 6}$$

Thus, re-arranging equation 6

$$e = \frac{P_a}{R_v}\frac{H}{3} = \frac{3.43 \times 0.4 \times 10^3}{0.665 \times 2000 \times 9.81 \times 1.2}$$

= 0.088 m

This is less than 100.8mm and therefore no tension occurs in the stem.
The shear stress at the bottom of the wall is:

$$\frac{3.43 \times 10^3}{665 \times 1000} = 0.005 \text{ N/mm}^2$$

This is less than 0.1 N/mm^2, and there fore satis factory.

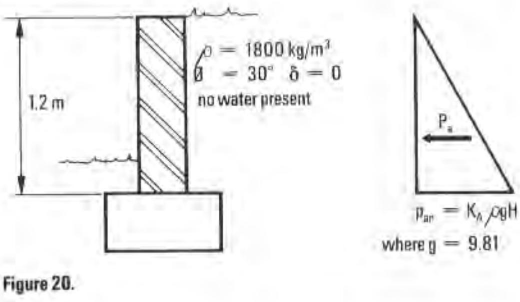


Figure 20.

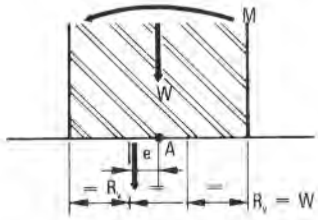


Figure 21.

Design of Base

Try a 300mm deep by 900mm wide base.

Find the nett eccentricity of the resultant at the underside of the base:

$$P_a = \frac{(1.2 + 0.3)^2}{2} \times 0.27 \times \frac{1800 \times 9.81}{1000}$$

$= 5.36$ kN/m

$$W_1 = 0.12 \times 1.2 \times \frac{1800 \times 9.81}{1000} = \quad 2.54 \text{ kN/m}$$

$$W_2 = 0.665 \times 1.2 \times \frac{2000 \times 9.81}{1000} = 15.66 \text{ kN/m}$$

$$W_3 = 0.9 \times 0.3 \times 24 = \quad 6.48 \text{ kN/m}$$

$$W_1 + W_2 + W_3 = \quad 24.68 \text{ kN/m}$$

See Figure 22 (opposite)

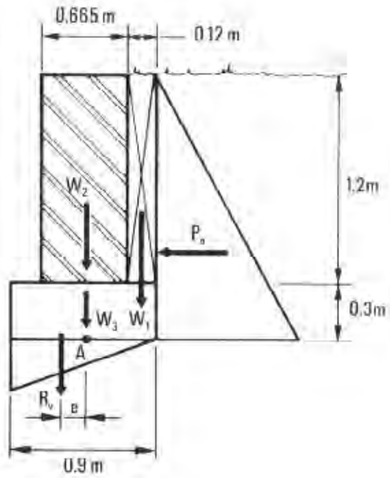


Figure 22.

Taking moments about A, neglecting passive pressure:

$$24.68e = 5.36 \times \left\{\frac{1.2 + 0.3}{3}\right\} - 2.54 \left\{\frac{0.665}{2} + \frac{0.12)}{2}\right\}$$

$$\text{Therefore, } e = \frac{2.68 - 1.00}{24.68} = \frac{1.68}{24.68} = 0.07\text{m}.$$

Thus, the resultant is within the middle third of the base, and therefore no uplift of the base will occur and there is an adequate factor of saftey against overturning.

The maximum bearing pressure under the toe is given by:

$$\frac{W}{A} + \frac{We}{Z} = \frac{24.68}{0.9 \times 1.0} + \frac{24.68 \times 0.07 \times 6}{1.0 \times 0.9^2}$$

$= 27.4 + 12.8 = 40.2$ kN/m^2.

This is less than the safe bearing capacity and therefore satisfactory.

The force causing sliding of the wall $= P_a = 5.36$ kN/m

The force resisting sliding = frictional force.

As the base is in situ, the frictional force,

$F_p = \tan ø$.

$W = 0.7 \times 24.68 = 17.28$ kN

This provides a factor of safety of $\frac{17.28}{5.36} = 3.2$ against sliding

The passive pressure could also have been included in this calculation, if necessary.

(ii) 0.07 N/mm^2 permissible stress allowed. The overturning moment, M, = 1.37 kNm/m, as in (i).

Design of stem

$$p_t = \frac{M}{Z} - \frac{W}{A} \quad \text{.......... 7}$$

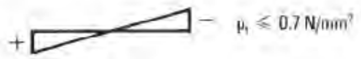


Figure 23 **Stress diagram at base of wall**

Using equation 7 and solving for t, the minimum thickness of wall required:

$$0.07 \times 10^3 = \frac{1.37 \times 6}{1.0t^2} - \frac{2000 \times 9.81 \times 1.2 \times t}{1000 \times t \times 1.0}$$

Therefore, $(70 + 23.54)t^2 = 8.22$

and $t = \sqrt{\frac{8.22}{93.54}} = \sqrt{0.09} = 0.3\text{m} = 300\text{mm}$

Therefore a 1½ brick wall required.

Thickness $= 3 \times 102.5 + 2 \times 10 = 327.5$mm.

The shear stress at the bottom of the wall ($P_a = 3.43$ kN/m as in (i))

$$= \frac{3.43 \times 10^3}{327.5 \times 1000} = 0.01 \text{ N/mm}^2$$

This is less than 0.1 N/mm^2 and therefore satisfactory.

The base design follows the same procedure as that given in (i).

(iii) Assume bricks having a water absorption of 10% are laid in 1:¼:3 mortar.

From Table 5, for bending about a horizontal axis, a permissible tensile stress of 0.11 N/mm^2 may be used.

The lateral force on the wall, $P_a = 3.43$ kN/m, and the overturning moment = 1.37kNm/m, as in (i).

Since $p_t = \frac{M}{Z} - \frac{W}{A}$

Then, $0.11 \times 10^3 = \frac{1.37 \times 6}{1.0 \times t^2} - \frac{2000 \times 9.81 \times t \times 1.2}{1000 \times t \times 1.0}$

Simplifying, $(110 + 23.54)t^2 = 8.22$.

Therefore, $t = \sqrt{\frac{8.22}{133.54}} = \sqrt{0.062}$

$= 0.25\text{m} = 250\text{mm}$

Thus a 1½ brick wall is required, thickness = 327.5mm.

The shear stress at the bottom of the wall

$= \frac{3.43 \times 10^3}{327.5 \times 1000} = 0.01\ \text{N/mm}^2$

This is less than 0.1 N/mm^2 and therefore satisfactory.

The base design again follows the same procedure as for (i).

Example 2

A reinforced brickwork retaining wall 4.0m high has to support a cohesionless soil of 1600 kg/m^3 density and having a $ø = 30°$. The bricks to be used have a crushing strength of 34.5 N/mm^2 laid in 1:¼:3 mortar (designation (i)). The reinforcement is high yield with a permissible tensile stress of 210 N/mm^2. The bearing capacity of the soil beneath the base is 100 kN/m^2.

See Figure 25 (opposite)

Friction between the back of the wall and the soil may be neglected, i.e. $\delta = 0$.

Lateral earth pressure on wall:
From Table 2, Section 3.1, the coefficient of active pressure
$K_A = 0.33$ for $ø = 30°$.

Thus, the total lateral load on stem

$P = K_A \rho g H \times \frac{H}{2}$

$= \frac{0.33 \times 1600 \times 9.81}{1000} \times \frac{4.0^2}{2} = 41.4\ \text{kN/m}$

hence moment at base of stem

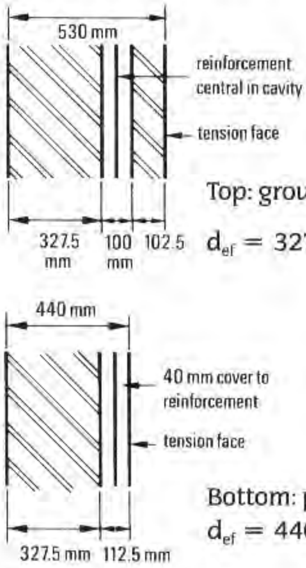
$= 41.4 \times \frac{4}{3} = 55.2\ \text{kNm/m}$

From Table 5, Q = 0.417 for 34.5 N/mm^2 bricks.
Hence, from equation 5, section 3.3.2(a):

$bd_{ef}^2 = \frac{55.2 \times 10^6}{0.417} = 1.32 \times 10^8$

Therefore; $d_{ef} = \sqrt{\frac{1.32 \times 10^8}{1000}} = 364\text{mm}$.

This may be obtained as follows;



Top: grouted cavity wall
$d_{ef} = 327.5 + \frac{100}{2} = 377.5\text{mm}$

Bottom: pocket wall
$d_{ef} = 440 - 40 - 10 = 390\text{mm}$

Figure 24

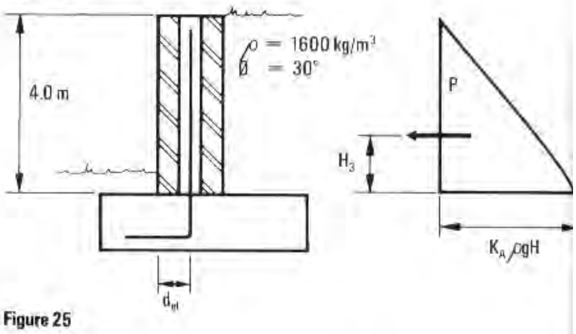


Figure 25

To obtain the area of reinforcement:
From Table 5, $a_1 = 0.908$
Hence, for grouted cavity wall:

$A_s = \frac{55.2 \times 10^6}{0.91 \times 377.5 \times 210} = 765\text{mm}^2/\text{m}$

and for pocket wall:

$A_s = \frac{55.2 \times 10^6}{0.91 \times 390 \times 210} = 741\ \text{mm}^2/\text{m}$

Grouted cavity wall; use 12mm diameter bars at 100mm centres, to give the minimum area of reinforcement, see Section 4.2.

Pocket wall; use pockets at 900mm centres each reinforced with 4 x 16mm diameter bars.

Choosing a grouted cavity wall;
Shear force on wall = 41.4 kN/m

therefore, shear stress $= \frac{41.4 \times 10^3}{530 \times 1000}$

$= 0.08\ \text{N/mm}^2$

Permissible shear stress = 0.1 N/mm^2. This exceeds the actual stress and, is therefore, satisfactory. Note, had a pocket wall been chosen, it would have been necessary to increase the thickness of the wall at the bottom to reduce the actual shear stress to the permissible value - see Example 3.

Design of Base

Assume base to be 350mm deep and length of toe to be 1.0m.

Resistance to sliding:

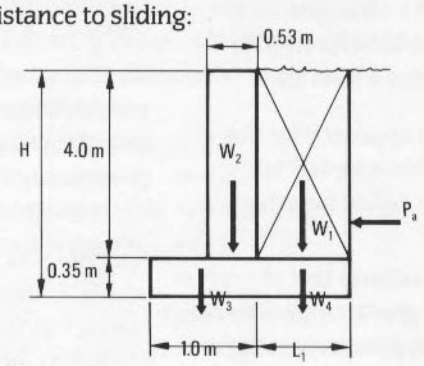


Figure 26

$$P_a = K_A \rho g \frac{H^2}{2} = \frac{0.33 \times 9.81 \times 1600}{1000} \times \frac{(4.0 + 0.35)^2}{2} = 49 \text{ kN/m}$$

$$W_1 = \frac{1600 \times 4.0 \times L_1 \times 9.81}{1000} = 62.8L_1 \text{kN/m}$$

$$W_2 = \frac{2000 \times 4.0 \times 0.53 \times 9.81}{1000} = 41.6 \text{ kN/m}$$

$W_3 = 1.0 \times 0.35 \times 24 \quad = 8.4 \text{ kN/m}$

$W_4 = 0.35 \times 24 \times L_1 \quad = \underline{8.4L_1 \text{kN/m}}$

Total W $\quad = 71.2L_1 \text{kN/m} + 50.0 \text{kN/m}$

As base is in situ, coefficient of friction $= \tan ø = 0.57$

Force causing sliding, P_a, = 49 kN/m

Thus, equating the resisting force to the lateral force, multiplied by a factor of safety of 2.0 and solving for L_1:

$49 \times 2.0 = (71.2L_1 + 50.0)0.57$

Hence, L_1 =1.71m, say 1.75m

Resistance to overturning:

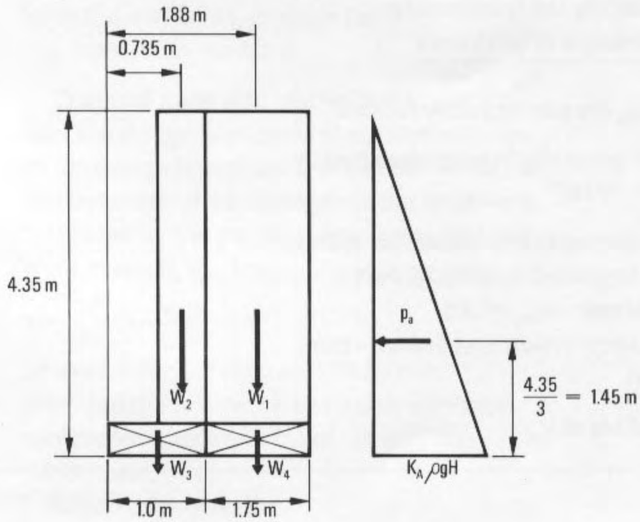


Figure 27

Taking moments about A, overturning moment $= 49 \times 1.45 = 71.1$ kNm/m.

Righting moment from

$W_1 = 62.8 \times 1.75 \times 1.88 = 206.6$

$W_2 = 41.6 \times 0.735 \quad = 30.6$

$W_3 = 8.4 \times 0.5 \quad = 4.2$

$W_4 = 8.4 \times 1.75 \times 1.88 = \underline{27.6}$

Therefore, total $\quad = 269.0$ kNm/m

thus, the factor of safety is $\frac{269.0}{71.1} = 3 \cdot 8$

This exceeds 2.0 (see section 3.2a) and is therefore satisfactory.

Ground bearing pressure:

total vertical load

$= 62.8 \times 1.75 + 41.6 + 8.4 + 8.4 \times 1.75$

$= 174.7$ kN/m,

nett moment on base about point A

$= 269.0 - 71.1 = 197.9$ kNm/m.

Thus, the resultant acts $\frac{197.9}{174.7} = 1.13$m from A.

Therefore, eccentricity $= 1.38 - 1.13 = 0.25$m,

and maximum ground pressure $= \frac{W}{A} + \frac{We}{Z}$

$$= \frac{174.7}{2.75 \times 1.0} + \frac{174.7 \times 0.25 \times 6}{1.0 \times 2.75^2}$$

$= 63.5 + 34.7 = 98.2 \text{ kN/m}^2$

This is less than permissible and therefore satisfactory,

Base reinforcement:

The design of the base is in accordance with CP114.

3.4 LIMIT STATE DESIGN

3.4.1. Principles

The limit state design philosophy is based upon the principle that there must be an acceptable probability that a structure, or any of its parts, will not become unfit for its purpose, i.e., that it will not reach a limit state.

The use of the limit state approach for the design of brickwork has been adopted in BS5628 in line with current policy for other structural codes.

BS5628:Part 1:1978, 'Structural Use of Unreinforced Masonry'[1], gives requirements for ensuring that there is an adequate margin of safety against the ultimate limit state being reached. Generally there will be an adequate margin of safety against the serviceability limit states of cracking (and deflection) being reached when the design satisfies the ultimate limit state.

BS5628:Part 2:1985, 'Structural Use of Reinforced and Prestressed Masonry' [2] defines three limit states to be examined. These are the ultimate limit state and the two serviceability limit states of deflection and cracking. In the case of reinforced brickwork cantilever retaining walls, the serviceability limit states are said to be satisfied by ensuring that the wall has a height to effective thickness ratio of not more than 18 when there is vertical reinforcement of up to 0.5% of the width times the effective depth of the brickwork section. For greater amounts of reinforcement, deflection should be checked. The method for this is beyond the scope of this document.

In order to ensure that the ultimate limit state is not reached, partial safety factors are applied to material strengths and loads, so that design must satisfy the relationship:

$$\gamma_f \times \text{load} \leqslant \frac{\text{strength of brickwork}}{\gamma_m}$$

where γ_f and γ_m are partial safety factors.

3.4.2. Mass brickwork retaining walls

Design will be generally in accordance with BS5628:Part 1:1978[1]

(a) Partial safety factors for materials:

The following partial safety factors for material strength are suggested in BS5628:Part 1
Brickwork in shear $\gamma_{mv} = 2.5$
Brickwork in compression and flexure - from Table 6 (below)

TABLE 6. Values of γ_m:

		category of construction control	
		special	normal
category of manufacturing control	special	2.5	3.1
	normal	2.8	3.5

The categories given in Table 6 may be interpreted as follows:

Category of construction control

Normal - this category should be assumed whenever the work is carried out in accordance with the recommendations for workmanship given in section four of BS5628:Part 3,[5] including appropriate supervision and inspection.

Special - this category may be assumed where, additionally to the 'normal' requirements, the specification and site supervision standards justify the lower partial safety factors given in Table 5 and preliminary and site testing of the mortar is carried out in accordance with Appendix A1 to the code.

Category of manufacturing control

Special - the manufacturer must agree to meet an 'acceptance limit' for the compressive strength of his bricks, i.e., not more than 2½% of the bricks may fall below this limit, and he must operate a quality control scheme enabling the 'acceptance limit' to be met.

Normal - the supplier can meet the compressive strength requirements for the bricks, but not the 'acceptance limit'.

(b) Partial safety factors for load

The partial safety factors for loads, γ_f, vary according to the load to which they are applied and the design case being considered.
The following values are given in BS5628:Part 1 for the dead and imposed load case.
Stem design
lateral earth loads = 1.4
water pressure = 1.4
vertical dead loads = 1.4 or 0.9
passive pressure = 1.4
imposed loads = 1.6
NB BS5628:Part 1 states that for earth retaining and foundation structures 'when applying γ_f, no distinction is made between adverse and beneficial loads'. Thus the same partial safety factor is used for all earth loads. However, the value on beneficial dead loads other than from earth loads remains 0.9.

(c) Safety factors for overall stability

BS5628:Part 1 suggests that design for overall stability should be carried out by appropriate geotechnical procedures, i.e., by using characteristic loads without the application of partial safety factors, such that the requirements of section 3.2 are satisfied with regard to overall factors of safety.

(d) Design of stem

For mass brickwork walls, the design moment due to the lateral load on the stem is resisted by the dead weight of the wall combined with the flexural strength of the brickwork.
The design moment is γ_f Px (see figure 28).

The design moment of resistance of the wall is given by:

$$\left\{\frac{f_{kx}}{\gamma_m} + \frac{\gamma_f W}{A}\right\}Z$$

which must be greater than $\gamma_f Px$
where f_{kx} is the characteristic flexural strength of brickwork for bending causing failure parallel to the bed joints, given in Table 7.
Z is the section modulus.

The stem must also be checked to ensure that the design horizontal shear stress, v, due to the design lateral loads does not exceed the characteristic shear strength of the brickwork, f_v, divided by the partial safety factor for brickwork in shear, γ_{mv}, thus:

$$v = \frac{\gamma_f P}{bt} \leqslant \frac{f_v}{\gamma_{mv}}$$

where f_v may be taken as 0.35 N/mm^2 provided the brickwork has a characteristic compressive strength (f_k) of not less than 7.0 N/mm^2.
t = thickness of wall
b = width of section considered, usually 1.0m.

(e) Design of base

As with the permissible stress approach, the base must be proportioned so that tension does not occur in the concrete, no uplift of the heel of the wall occurs, and the safe bearing capacity of the soil is not exceeded. If a reinforced concrete base is used it should be designed in accordance with BS8110:1985[11], Structural use of concrete.

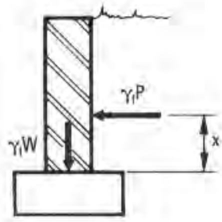
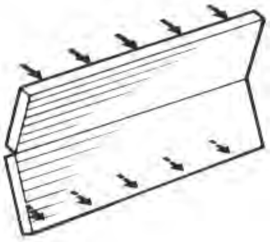
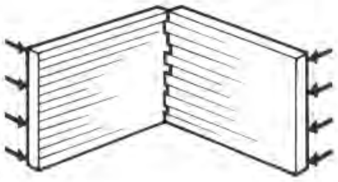


Figure 28.

TABLE 7. Characteristic flexural strength and permissible tensile stress for brickwork





direction of bending	type of brick		characteristic flexural strength f_{kx} (N/mm²) mortar designation (i)	(ii)	permissible tensile stress $p_t = \frac{f_{kx}}{4.5}$ (N/mm²) mortar designation (i)	(ii)
about horizontal axis	clay with water absorption	less than 7%	0.7	0.5	0.15	0.11
		between 7% and 12%	0.5	0.4	0.11	0.09
		between 12% and 30%	0.4	0.3	0.09	0.07
parallel to bed joints	calcium silicate		0.3		0.07	
about vertical axis	clay with water absorption	less than 7%	2.0	1.5	0.44	0.33
		between 7% and 12%	1.5	1.1	0.33	0.24
		between 12% and 30%	1.1	0.9	0.24	0.2
perpendicular to bed joints	calcium silicate		0.9		0.2	

Mortar (i) is 1:0–¼:3 cement:lime:sand
Mortar (ii) is 1:½:4–4½ cement:lime:sand
The values of f_{kx} and p_t for the mode of failure perpendicular to the bed joints are used only where unreinforced brickwork spans partly or wholly horizontally. e.g. where the wall is buttressed.

3.4.3 Reinforced brickwork retaining walls

Design will be generally in accordance with BS5628: Part 2

(a) Partial safety factors for loads:

The partial safety factors for loads, γ_f, used in the design of reinforced brickwork are the same as those in the design of unreinforced brickwork, as given in 3.4.2 (b), since ideally they should be independent of whatever structural material is being used.

(b) Partial safety factors for materials.

BS5628: Part 2 gives partial safety factors for each aspect of material strength which affects the strength of reinforced brickwork, namely, γ_{mm} for compressive strength of brick, γ_{mv} for shear strength of brickwork, γ_{mb} for bond strength between infill concrete or mortar and reinforcement, and γ_{ms} for strength of reinforcement.

Different values are given in the code for the ultimate and serviceability limit states. However, as no calculations are required for the serviceability limit states of deflection and cracking for cantilever retaining walls, provided the height to effective thickness ratio and percentage area of reinforcement, given in 3.4.1, are not exceeded, only values for the ultimate limit state are given here.

The values of γ_{mm} assume that all the recommendations given in clause 40.1 and 40.2 of BS5628: Part 2, dealing with the quality control of workmanship and materials, will be followed. These recommendations correspond generally to the special category of construction control referred to in 3.4.2 and include frequent visits to site by the designer or the presence of his permanent representative on site and preliminary and site testing and sampling of bricks, mortar and infill concrete.

The normal and special categories of manufacturing control are as for unreinforced masonry (see 3.4.2).

If these recommendations cannot be met then the values of γ_{mm} etc given in Tables 8 and 9 below must be increased accordingly.

TABLE 8. **Partial Safety Factors, γ_{mm}, for reinforced brickwork in direct compression and bending; ultimate limit state.**

Category of Manufacturing Control (see 3.4.2)	Value of γ_{mm}
special	2.0
normal	2.3

TABLE 9 Partial Safety Factors, γ_{mv}, γ_{mb} and γ_{ms} for reinforced brickwork.

Partial Safety Factor	Value
γ_{mv}, shear strength of brickwork	2.0
γ_{mb}, bond strength between concrete infill or mortar and reinforcement	1.5
γ_{ms}, strength of reinforcement	1.15

(c) Safety factors for overall stability

As for unreinforced brickwork retaining walls, the requirements of section 3.2 should be satisfied, using characteristic loads throughout to calculate overall factors of safety.

(d) Design of Stem

For reinforced brickwork retaining walls, the moment due to the design lateral load is resisted by the stem of the retaining wall in flexure. Equation 9 given below for the design moment of resistance of brickwork in flexure is taken from BS5628:Part 2 and is based on the strain diagram and stress block diagram shown in Figure 29. The strain compatibility method is used for its derivation. The depth to the neutral axis, x_n, is calculated by equating the compressive and tensile forces. The value of y is then found and, thus, the lever arm, z, from:

$$z = d - y = d\left\{1 - \frac{0.5\,A_s f_y \gamma_{mm}}{\gamma_{ms} b d f_k}\right\} \leqslant 0.95d \quad \ldots\ldots 8$$

where f_y is the characteristic tensile strength of the reinforcement given in Table 10 and f_k is the characteristic compressive strength of brickwork given in Table 11.

The design moment of resistance of the wall, based on the strength of the brickwork is given by:

$$M_d = \frac{0.4\,f_k\,bd^2}{\gamma_{mm}} \quad \ldots\ldots 9$$

The design moment of resistance of the wall, based on the reinforcement is:

$$M_d = \frac{A_s f_y z}{\gamma_{ms}} \quad \ldots\ldots 10$$

The required value of A_s may be obtained iteratively by initially assuming $Z = 0.75d$, substitution of this value of z into equation 10 enables a value of A_s to be calculated which is then substituted back into equation 8 to obtain a more accurate value of z. This process is repeated until no further iteration is required.

A_s may be calculated explicitly from the following equation:

$$A_s = \frac{bd\,f_k\,\gamma_{ms}}{f_y\,\gamma_{mm}}\left\{1 \pm \sqrt{1 - \frac{2\gamma_{mm}M}{bd^2 f_k}}\right\} \quad \ldots\ldots 11$$

In both cases it is necessary to check that z does not exceed 0.95d and that the design moment does not exceed the value of M_d obtained from equation 9.

Alternatively, the design chart given in BS5628:Part 2 may be used.

In the case of pocket type walls, the brickwork between the pockets is assumed to behave as a flange to the reinforced section. The thickness of the flange, t_f, may be taken as equal to the thickness of the brickwork up to a maximum of 0.5d. The width of the flange may be taken as the least of:

a) the spacing of the pockets

b) the width of the pocket plus 12 times the flange thickness

c) one third of the wall height

The design moment of resistance is calculated in the same way as for a continuously reinforced wall, but it must not exceed

$$M_d = \frac{f_k}{\gamma_{mm}} b t_f\,(d - 0.5t_f) \quad \ldots\ldots 12$$

Where the spacing of the pockets exceeds 1 m, the ability of the masonry to span horizontally between them should be checked.

The shear stress, v, in the stem is calculated from

$$v = \frac{V}{bd}$$

Where V is the shear force due to design loads, i.e. in the case of cantilever retaining walls, $\gamma_f \times P$ (total design active lateral load).

v must be less than the shear strength of the stem,

$$\frac{f_v}{\gamma_{mv}}$$

where f_v is the characteristic shear strength of brickwork.

Where the reinforcement is wholly surrounded in mortar, f_v is taken as 0.35 N/mm^2: where it is wholly surrounded by infill concrete $f_v = 0.35 + 17.5\rho$ provided it does not exceed 0.7 N/mm^2.

Where $p = A_s/bd$

If shear resistance is inadequate, shear reinforcement may be provided or the thickness of the section increased, assuming that in the case of reinforcement surrounded by concrete, increasing A_s does not permit a sufficient increase in f_v.

The normal solution will be to increase the thickness of the section, as shear reinforcement is not easy to incorporate.

(e) Design of Base

The reinforcement in the base should be designed to BS8110 so that the design bending moment at the face of the retaining wall, due to the nett ground pressure, represented by the ground pressure diagram in Figure 19, is less than the design moment of resistance.

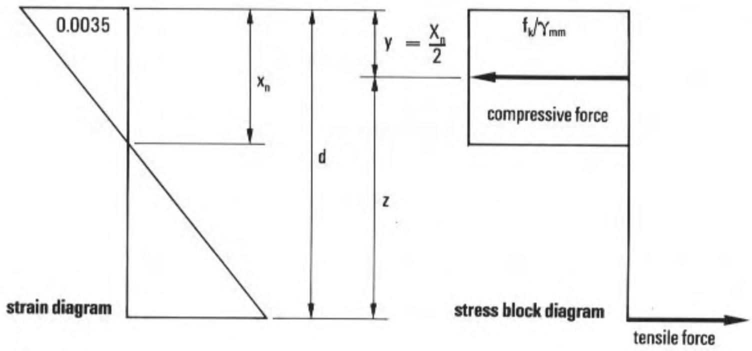


Figure 29. Limit state analysis

TABLE 10. Characteristic tensile strength of reinforcing steel, f_y

Designation	Nominal Size	Characteristic tensile strength, f_y
		N/mm^2
Hot rolled plain steel bars complying with BS4449	All	250
Hot rolled deformed high yield steel bars complying with BS4449	All	460
Cold worked steel bars complying with BS4461	All	460
Hard drawn steel wire complying with BS4482 and steel fabric complying with BS4483	up to and including 12	485
Stainless steel complying with BS970: Part 1 grades 304S15,316S31 or 316S33	All	460

TABLE 11 Characteristic compressive strength, f_k, of brickwork

(A) Constructed with bricks or other units having a ratio of height to least horizontal dimension of 0.6

Mortar designation	Characteristic compressive strength of brickwork $f_k (N/mm^2)$								
	Compressive strength of brick (N/mm^2)								
	7	10	15	20	27.5	35	50	70	100
(i)	3.4	4.4	6.0	7.4	9.2	11.4	15.0	19.2	24.0
(ii)	3.2	4.2	5.3	6.4	7.9	9.4	12.2	15.1	18.2

Mortar (i) is 1:0 — $\frac{1}{4}$:3 cement:lime:sand Mortar (ii) is 1: $\frac{1}{2}$:4 — $4\frac{1}{2}$ cement:lime:sand

3.4.4 Example 3

The brick dimensions used in this example are work sizes as defined in BS3921:1985[10]

A vertical grouted cavity brickwork retaining wall, using 50.0 N/mm^2 bricks in $1:\frac{1}{4}:3$ mortar, 4.0m high, supports partially saturated soft clay soil, the surface of which is level with the top of the wall. Assume special control of brick manufacture and normal control of construction. The saturated density of the soil is 1950 kg/m^3, its shear strength at zero normal load is 25 kN/m^2 and its angle of internal friction ø, is 5°. The soil beneath the wall has a safe bearing capacity of 150 kN/m^2. The water table is 3m below the top of the wall. Design the wall, neglecting wall adhesion.

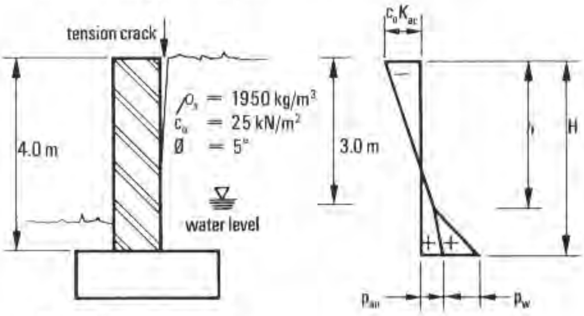


Figure 30

Lateral pressure

From CP2[6], the horizontal pressure at the base of the wall

$$= p_{an} - p_w = [\rho_s h + \rho_b(H-h)]K_A - c_OK_{AC} + \rho_w(H-h)$$

Where p_{an} = horizontal component of active earth pressure

p_w = water pressure

ρ_s = saturated density of soil

h = depth of water table below top of wall

ρ_b = submerged density of soil

K_A = cohesive soil coefficient, see Table 5 CP2

K_{AC} = cohesive soil coefficient, see Table 5 CP2

c_o = shear strength at zero normal load

H = height of wall

ρ_w = density of water

From Table 5, CP2:

K_A = 0.85 for zero adhesion

K_{AC} = 1.83 for zero adhesion

and
$$p_{an} = \left\{\frac{1950 \times 9.81 \times 3.0}{1000} + \frac{950 \times 9.81(4-3)}{1000}\right\} 0.85 - 25 \times 1.83$$
$$= 11.0\ kN/m^2$$

This is less than 4.8H which equals 19.2 kN/m^2 (see section 3.1)

Therefore use an active pressure of 19.2 kN/m^2

$$p_w = \frac{1000 \times 9.81}{1000}(4-3) = 9.81\ kN/m^2$$

The lateral loads:

$$P_a = 19.2 \times \frac{4}{2} = 38.4kN/m$$

$$P_w = 9.81 \times \frac{(4-3)}{2} = 4.9kN/m$$

$$= 43.3kN/m$$

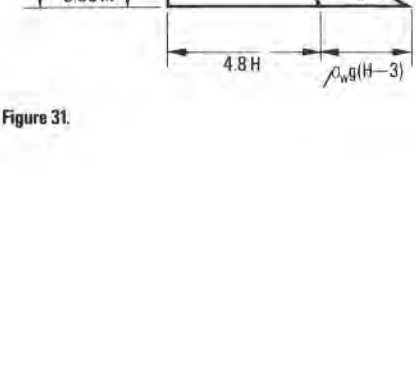


Figure 31.

(a) Design of Stem

The design lateral forces

$\gamma_f P_a \quad = 1.4 \times 38.4 = 53.8$ kN/m

$\gamma_f P_w \quad = 1.4 \times 4.9 \quad = \underline{\ 6.9}$ kN/m

and P = 60.7 kN/m

Therefore, the design moment, M, =
$53.8 \times 1.33 + 6.9 \times 0.33 = 73.8$ kNm/m.
From BS5628:Part 2, Table 8: the maximum H/d = 18

thus, the minimum d $= \dfrac{4000}{18} = 222$mm

Assume a one brick leaf (thickness 215mm) on the compression face of the wall, and high tensile stainless steel, 20mm diameter, reinforcement placed to give 20mm (min) cover. This arrangement gives a d of 285.0mm.

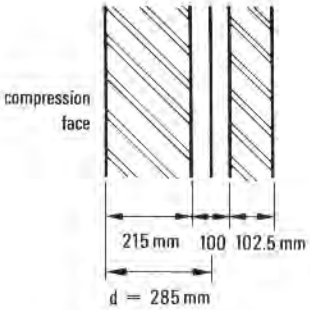


Figure 32.

From Table 11, for 50 N/mm² bricks in 1:¼:3 mortar, $f_k = 15$ N/mm²
From Table 8, $\gamma_{mm} = 2.0$

Using equation 9, Section 3.4.4(a), the design moment of resistance

$$M_d = \frac{0.4 \times 0.285^2 \times 15.0 \times 10^3}{2.0}$$

$= 243.7$ KNm/m

This exceeds the design moment and is therefore satisfactory.

From Table 9, $\gamma_{mv} = 2.0$

$\gamma_{ms} = 1.15$

Using equation 11, Section 3.4.3 (d), the area of reinforcement

$$A_s = \frac{285 \times 15 \times 1.15}{460 \times 2.0} \times \left\{ 1 - \sqrt{1 - \frac{2 \times 2.0 \times 73.8 \times 10^6}{285^2 \times 15 \times 10^3}} \right\}$$

$= 0.693$mm²/mm run

$= 693$ mm²/m run

Use 16mm diameter high tensile reinforcing bars at 250mm centres, giving 804mm²/m = 0.37% of bd. Use 10mm diameter high tensile distribution bars at 300 mm centres giving 261 mm²/m = 0.09% of bd.

Shear:
The design shear force on the base of the wall is equal to the total lateral load on the wall = P = 60.7 kN/m.

The shear stress due to design loads

$$v = \frac{60.7 \times 10^3}{1000 \times 285} = 0.21 \text{ N/mm}^2$$

The design shear strength of the wall =

$$\frac{0.35 + \left\{ 17.5 \dfrac{A_s}{bd} \right\}}{\gamma_{mv}} =$$

$$\frac{0.35 + \left\{ 17.5 \dfrac{804}{1000 \times 285} \right\}}{2.0} = 0.2\text{N/mm}^2$$

As the design shear force exceeds this, either a higher design shear strength must be made available or the effective depth of the wall must be increased. Assuming no increase in the design shear strength, the effective depth d, to the reinforcement must be increased by thickening the wall. The minimum d required for a design shear strength of 0.2 N/mm²

$$= \frac{60.7}{1000 \times 0.2} = 0.304\text{m} = 304\text{mm}$$

Therefore the thickness should be increased by half a brick to give a d of 407mm.

The design shear force reduces rapidly as the depth below retained ground level reduces and so the thickness of the wall may be reduced accordingly. Check the design shear force at 3.85 below the top of the wall:

$$= \frac{4.8 \times 3.85^2}{2} \times 1.4 + \frac{1000 \times 9.81}{1000} \times \frac{0.85^2}{2} \times 1.4 = 49.8 + 5.0 = 54.8 \text{ kN/m}$$

$$\text{Therefore } v = \frac{54.8 \times 10^3}{1000 \times 285} = 0.19 \text{ N/mm}^2$$

This is less than the design shear stress in the main wall; therefore the additional half brick may be stepped back at this level as shown in Figure 33.

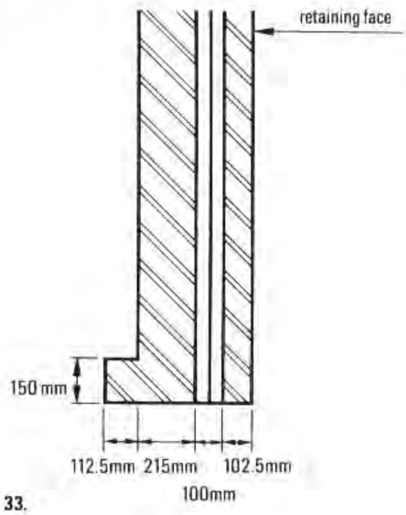


Figure 33.

Alternatively the design shear strength of the wall can be increased by increasing the amount of tension reinforcement.

Increased area of reinforcement required to provide design shear strength of 0.21 N/mm²:

$$A_s = (0.21 \times 2.0 - 0.35)\frac{1000 \times 285}{17.5}$$

$$= 1140\text{mm}^2/\text{m}$$

Therefore increase reinforcement to 16mm diameter bars at 175mm centres giving 1150mm²/m.

(b) Design of Base

Assume that a 400mm deep base will be adequate for bending, the design of the base reinforcement being to BS8110. For the purposes of this section, the thickening required to meet the shear requirements has been ignored.

Heel length - sliding analysis
(assume toe length = 1.2m)

$$P_a = \frac{4.8 \times 4.4^2}{2} = 46.5 \text{ kN/m}$$

$$P_w = \frac{1000 \times 9.81}{1000} \times \frac{1.4^2}{2} = 9.6 \text{ kN/m}$$

Therefore, force causing sliding =
46.5 + 9.6 = 56.1 kN/m.

To provide an adequate factor of safety against failure by sliding, the force resisting sliding must exceed the force causing sliding by a factor of at least 2.0.
Force resisting sliding
= Area of base x adhesion (25 kN/m²)
= $(1.2 + L_1)$ 1.0 x 25.
Equating these forces, including the factor of safety, and solving for L_1:

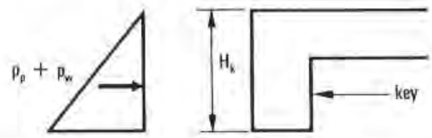
$$L_1 = \frac{2.0 \times 56.1 - 1.2 \times 25}{25} = 3.3\text{m}$$

This is excessive, therefore provide a key to the base and consider passive pressure.
Make heel 2.0m long. Therefore the passive force required, neglecting the ground above the toe =
2.0 x 56.1 - (1.2 + 2.0) x 25 = 32.2kN/m.

Assume water level is at the top of the base (i.e., a waterproof slab over the base and natural drainage prevents it rising higher). Therefore, from CP2[(6)]:

$$p_p + p_w = K_p\rho_b g H_k + K_{pc}C_o + \rho_w g H_k$$

$$\text{and } P_p + P_w = K_p\rho_b g\frac{H_k^2}{2} + K_{pc}C_o H_k + \rho_w g\frac{H_k^2}{2}$$



and from Table 7, CP2, for zero adhesion:
K_p = 1.2
K_{pc} = 2.2

Therefore,

$$\frac{1.2 \times 950 \times 9.81}{1000}\frac{H_k^2}{2} + 25 \times 2.2 \times H_k + \frac{1000 \times 9.81}{1000}\frac{H_k^2}{2} \geqslant 32.2$$

Re-arranging and solving for H_k:
H_k = 0.53m.
Therefore make toe project 0.15m below base.
Resistance to overturning
Overturning analysis: factor of safety against overturning of 2.0 required. Use characteristic loads.

Taking moment about C (see figure 34)

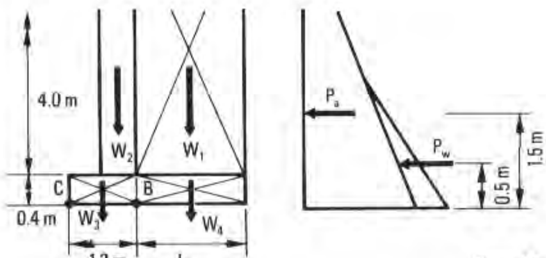


Figure 34

Load (kN/m)				Lever arm(m)	Moment about C(kNm/m)	
W_1 =	$\frac{4.0 \times 1950 \times 9.81 \times 2.0}{1000}$	=	153.0	+ 2.2	+ 336.7	
W_2 =	$\frac{4.0 \times 0.42 \times 2000 \times 9.81}{1000}$	=	33.0	+ 0.99	+ 32.6	
W_3 =	1.2 x 0.4 x 24	=	11.5	+ 0.6	+ 6.9	
W_4 =	2.0 x 0.4 x 24	=	19.2	+ 2.2	+ 42.2	
P_a =	46.5	=		− 1.5		− 69.8
P_w =	9.6	=		− 0.5		− 4.8
			215.7		+ 418.4	− 74.6

Factor of safety against overturning =

$$\frac{418.4}{74.6} = 5.6$$

Therefore the toe length chosen is satisfactory.

Ground bearing pressure

For both horizontal and vertical loads use characteristic loads. Taking moments about B (see Figure 36)

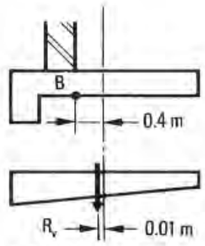


Figure 36.

Load (kN/m)			Lever arm(m)	Moment about B(kNm/m)	
W_1 =	153.0		+ 1.0	+ 153.0	
W_2 =	33.0		− 0.21		− 6.9
W_3 =	11.5		− 0.6		− 6.9
W_4 =	19.2		+ 1.0	+ 19.2	
P_a =		46.5	− 1.5		− 69.8
P_w =		9.6	− 0.5		− 4.8
$W = R_v$ =	216.7			+ 172.2	− 88.4

Therefore, the nett moment on the base = 172.2 − 88.4 = 83.8 kNm/m

Thus the vertical component of the resultant, R_v acts:

$$\frac{83.8}{216.7} = 0.39\text{m to the right of B.}$$

Eccentricity = 0.4 − 0.39 = 0.01 m, R_v is therefore within the middle third of the base and there will be no uplift of the heel.

The maximum bearing pressure =

$$\frac{\sum W}{\text{Area of base}} + \frac{\text{nett moment}}{Z}$$

$$= \frac{216.7}{3.2 \times 1.0} + \frac{216.7 \times 0.01 \times 6}{1.0 \times 3.2^2} = 69.0\ \text{kN/m}^2$$

This is less than 150 kN/m² and, therefore, satisfactory.

Design of base reinforcement

The base reinforcement is designed in accordance with BS 8110[11].

The maximum bending moments on the base are obtained from the bearing pressure diagram modified by the appropriate γ_f values as follows (see Table 2.1, BS 8110):

Case A

Vertical beneficial deadloads	$\gamma_f = 1.0$
lateral earth and water pressure	$\gamma_f = 1.4$

Note that no distinction is made between adverse and beneficial earth and water pressure in the value of γ_f.

Taking moments about C (see Figure 34)

Design Load (kN/m)			Lever arm(m)	Moment about B (kNm/m)	
W_1 =	1.4 x 153.0	= 214.2	+ 2.2	+ 471.2	
W_2 =	1.0 x 33.0	= 33.0	+ 0.99	+ 32.7	
W_3 =	1.0 x 11.5	= 11.5	+ 0.6	+ 6.9	
W_4 =	1.0 x 19.2	= 19.2	+ 2.2	+ 42.2	
P_a =	1.4 x 46.5	= 65.1	− 1.5		− 97.7
P_w =	1.4 x 9.6	= 13.4	− 0.5		− 6.7
		= 277.9		+ 553.0	− 104.4

$W = R_v = 214.2 + 33.0 + 11.5 + 19.2$
$= 277.9$ kN/m

Therefore vertical component of resultant,
R_v, acts: $\dfrac{553.0 - 104.4}{277.9} = 1.61$ m
from A, giving an eccentricity of 0.01 m, and a pressure diagram as shown in Figure 37

See Figure 37 (opposite)

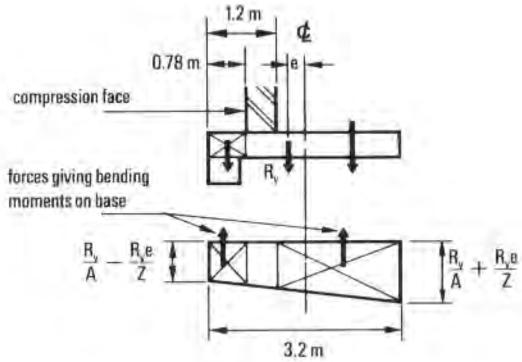


Figure 37.

Thus the maximum design pressure

$$= \frac{277.9}{3.2 \times 1.0} + \frac{277.9 \times 0.01}{1.0 \times 3.2^2} \times 6$$

$= 86.8 + 1.6 = 88.4$ kN/m^2

and the minimum pressure
$= 86.8 - 1.6 = 85.2$ kN/m^2.
The pressure at the compression face of the wall, from similar triangles,

$$=85.2 + \frac{(88.4 - 85.2)(3.2 - 2.24)}{3.2}$$

$= 86.0$ kN/m^2

Therefore, the bending moment for which the toe must be designed

$$= 85.2 \times \frac{0.78^2}{2} + (86.0 - 85.2) \times \frac{0.78^2}{2} \times \frac{1}{3} - \frac{0.78^2}{2} \times 0.4 \times 24 = 25.9 + 0.1 - 2.9$$

$= 23.1$ kNm/m.

The bearing pressure at the tension face of the wall

$$= 85.2 + \frac{(88.4 - 85.2)(3.2 - 2.0)}{3.2}$$

$= 86.4$ kN/m^2

Thus,the bending moment for which the heel must be designed

$$= (214.2 + 19.2)\,1.0 - 86.4 \times \frac{2.0^2}{2} - (88.4 - 86.4)\,\frac{2.0^2 \times 2}{2 \times 3}$$

$= 233.4 - 172.8 - 2.7 = 57.9$ kNm/m.

Case B

For vertical loads $\gamma_f = 1.4$.
Taking moments about C

Load (kN/m)			Lever arm(m)	Moment about B (kNm/m)	
$W_1 =$	1.4×153.0	$= 214.2$	$+ 2.2$	$+ 471.2$	
$W_2 =$	1.4×33.0	$= 46.2$	$+ 0.99$	$+ 45.7$	
$W_3 =$	1.4×11.5	$= 16.1$	$+ 0.6$	$+ 9.7$	
$W_4 =$	1.4×19.2	$= 26.9$	$+ 2.2$	$+ 59.2$	
$P_a =$	$1.4 \times 46.5 = 65.1$		$- 1.5$		$- 97.7$
$P_w =$	$1.4 \times 9.6 = 13.4$		$- 0.5$		$- 6.7$
$W = R_v =$		$= 303.4$		$+ 585.8$	$- 104.4$

Therefore, R_v acts

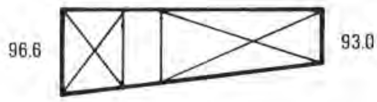
$$\frac{585.8 - 104.4}{303.4} = 1.59\text{m from A}$$

and this is within the middle third of the base.
Eccentricity of R_v, e, $= 1.6 - 1.59 = 0.01$ m.

Thus, maximum pressure

$$= \frac{303.4}{3.2 \times 1.0} + \frac{303.4 \times 0.01}{1.0 \times 3.2^2} \times 6$$

$= 94.8 + 1.8 = 96.6$ kN/m^2
and minimum pressure $= 94.8 - 1.8$
$= 93.0$ kN/m^2



At the compression face, from similar triangles, the pressure

$$= 93.0 + \frac{(96.6 - 93.0) \times (3.2 - 0.78)}{3.2}$$

$= 95.7$ kN/m^2
Therefore, the bending moment for which the toe must be designed

$$= (96.6 - 95.7)\ \frac{0.78^2}{2} \times \frac{2}{3} + 95.7 \times \frac{0.78^2}{2} - \frac{0.78^2}{2} \times 0.4 \times 24$$

$= 0.2 + 29.1 - 2.9 = 26.5$ kN/m^2

Bearing pressure at the tension face of the wall

$$= 93.0 + \frac{(96.6 - 93.0)(3.2 - 1.2)}{3.2}$$

$= 95.3$ kN/m^2

Thus, the bending moment for which the heel must be designed

$= (26.9 + 214.2)\ 1.0 - 93.0 \times 2.0 \times 1.0$

$$- (95.3 - 93.0)\ \frac{2.0^2}{2 \times 3}$$

$= 241.1 - 186.0 - 1.5 = 53.6$ kNm/m.

The maximum bending moment for the toe is due to Case B: M = 26.4 kNm/m.

The maximum bending moment for the heel is due to Case A: M = 57.9 kNm/m.

These moments should be used to calculate the areas of reinforcement required in the base.

4.0 DETAILS AND CONSTRUCTION

4.1 DURABILITY

Brickwork is a generally durable and aesthetically pleasing material. In order to maintain this durability and attractive appearance, it is important in the case of retaining walls, as in other types of walls, to prevent continuous saturation by good detailing.

Waterproofing details to prevent saturation (see figure 38) should include the following:
(a) a damp proof course capable of transmitting tension, e.g. damp proof course 2 clay brickwork as described in BS 3921 [10] just above the lower ground level.
(b) an effective waterproofing treatment on the retaining face of the wall, extending at least 150mm above the finished level of the retained material
(c) an effective coping which throws water clear of the exposed surfaces of the wall if clay bricks of moderate frost resistance (M) to BS 3921 are to be used. If frost resistant clay bricks (F) to BS 3921 are specified, either a coping or capping detail may be used.
(d) an effective damp proof course below the coping or capping. The dpc used should have good bonding characteristics to mortar and should be bedded on both sides.

The waterproofing treatment to the retaining face may consist of a spray or brush applied bituminous compound or an impervious sheet. In most cases, the former is more suitable. The lining should be protected against damage from the retained material both as the fill is placed and afterwards as it settles.

Provided that the above details are incorporated, a retaining wall may be constructed in clay bricks of moderate frost resistance (M) or calcium silicate bricks of Class 3 or stronger to BS 187 [12]. Where there is a risk of saturated brickwork becomimg frozen - for instance in an area with a high driving rain index - either frost resistant clay bricks (F), or calcium silicate bricks of Class 4 or stronger should be used. Where moderately frost resistant clay bricks are proposed for use in such situations, the manufacturer's advice should be sought.

Where aggressive sulphate ground conditions are known to exist, sulphate - resisting cement in a mortar of designation (i) (see Table 7) should be used up to and including the ground damp proof course, and above if the retaining face is not efficiently waterproofed. In areas with a high driving rain index, when clay bricks of normal soluble salt content (N) are used, mortar with sulphate-resisting cement should be used throughout the wall. The use of clay bricks with low soluble salt content (L) is preferred in these areas.

The choice of mortar designation to be used should be made on the basis of structural and durability considerations. It should be noted that engineering brick damp proof courses are laid in mortar of designation (i), and it is therefore advisable to use the same mortar for brickwork below that level. Where brick-on-edge copings are used, it will normally be necessary to use mortar designation (i) for durability.

To prevent the build up of water pressure behind a retaining wall, a land drain should be provided near the base of the stem on the retaining side. The drain must be surrounded by a granular, free-draining material preferably extending to the surface. Weep holes, if acceptable, should also be provided through the wall near the base level and at intermediate levels if necessary (see Figure 38 & 39).

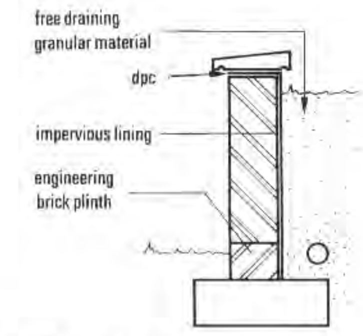


Figure 38. Waterproofing

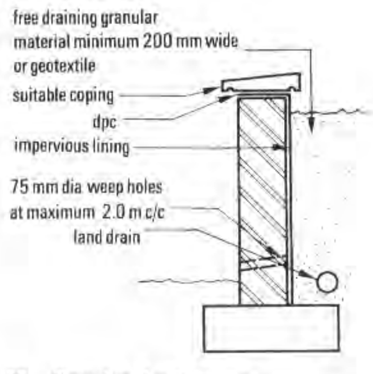


Figure 39. Prevention of water pressure behind wall

4.2 REINFORCED WALLS

(a) Area of main reinforcement

A minimum of main reinforcement is not defined in BS5628:Part 2[2]; however, the designer is advised to consider whether design to BS5628:Part 1[1] would be more appropriate when only small areas of reinforcement are required. When dealing with cantilever reinforced brickwork retaining walls it may be considered sensible to provide at least 0.15% bd for high yield reinforcement and 0.2% bd for mild steel reinforcement.

(b) Area of secondary reinforcement

A minimum area of 0.05% bd secondary reinforcement is required in reinforced brickwork retaining walls, except in the case of pocket walls. Secondary reinforcement may be omitted from these walls unless it is specifically required to tie the concrete infill to the masonry, as may be the case, for example, where contact areas between the concrete and masonry are relatively small.

(c) Spacing and size of bars

The minimum spacing between parallel bars should be the greater of:

(i) The maximum size of aggregate plus 5mm.
(ii) The bar diameter.
(iii) 10mm.

The maximum spacing of main and secondary tension reinforcement should not be greater then 500mm in grouted cavity and Quetta bond walls. In Quetta bond or other special bond walls only one bar, except at lap positions, is permitted in each core.

In grouted cavity and Quetta bond walls a maximum bar size of 25mm diameter is permitted by BS5628:Part 2. In pocket walls this maximum is increased to 32mm diameter. Reinforcement located in the bed joints should not exceed 6mm diameter.

(d) Cover

Brickwork retaining walls will generally be classified under exposure situation E3 in BS5628:Part 2. Where carbon steel reinforcement is used, the cover given in Table 12 should be provided for this exposure position.

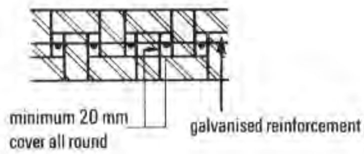
TABLE 12 Minimum concrete cover for carbon steel reinforcement

Concrete grade	30	35	40
Cover (mm)	40	35	30

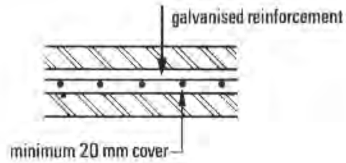
In grouted cavity or Quetta bond construction BS5628:Part 2 suggests that galvanised (to BS729[13], minimum coating mass 940g/m^2) carbon steel should be used for exposure situations E3 requiring a cover of 20mm of concrete or mortar. Alternatively stainless steel, or stainless steel coated, reinforcement can be used in which case there is no minimum cover requirement other than that required for the satisfactory development of bond. Where the water absorption of the bricks exceeds 10% only stainless steel reinforcement should be used.

Where bed joint reinforcement is required in situation E3 it should be stainless steel, or stainless steel coated, reinforcement, with a mortar cover of 15mm to the exposed face of the masonry. It would be normal, where bed joint reinforcement is used, to use stainless steel reinforcement for main steel also, to avoid bi-metallic corrosion.

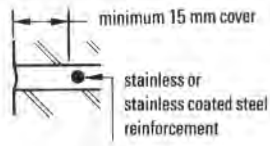
For details see Figure 40.



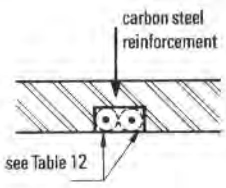
quetta bond



grouted cavity



bed joint



pocket wall

Figure 40. Cover to reinforcement

4.3 MOVEMENTS JOINTS

BS5628:Part 3[5] suggests that vertical movement joints should be provided in long runs of clay brickwork walling at a maximum of 15m centres and in calcium silicate brickwork at a maximum of 10m centres. As retaining walls are less susceptible to horizontal movements than other walls these values may be taken as appropriate for unreinforced brick retaining walls. The provision of horizontal reinforcement in the bed joints, or in a grouted cavity, will tend to restrain movement, and providing careful consideration is given to the type of brick and the design, the distance between joints may be increased, although no guidance can be given here as to the extent of the increase. When brick on edge copings are used, movement joints should be provided at about half the normal centres. A water bar, included in the joint, will help prevent staining of the face of the wall.

4.4 CONSTRUCTION

The maximum height of masonry that should normally be built in a day is 1.5m. It is especially important in reinforced brick walls that cavities and pockets, etc., are carefully cleared of any mortar droppings.

4.4.1 Grouted-cavity construction

Grouted cavity walls may be constructed either by the low-lift or high-lift methods. In the former the concrete infill is placed during the brick laying process at maximum vertical intervals of 450mm. Care must be taken to avoid damage to the wall resulting from excessive pressure from the infill, during the filling process. In the latter, high lift method, the concrete infill is placed not sooner than 3 days after the building of the wall to a maximum of 3m in height.

4.4.2 Quetta bond and similar walls

Usually the voids around the reinforcement are filled with mortar or concrete infill as the brickwork proceeds. If, however, the voids are sufficiently large the low-lift or high-lift methods as described above, may be used.

4.4.3 Pocket-type walls

The brickwork is usually built to full height before the infill concrete is placed. Care should be taken to ensure that the formwork to the pocket is adequately propped or tied to the brickwork to avoid disturbance during the filling and compacting processes

5.0 ECONOMY OF BRICKWORK RETAINING WALLS

There is general agreement that when brickwork is chosen as the cladding material, loadbearing brickwork can be the most economic form of structure for many building types. This has been based on solid experience as opposed to cost studies. For more detailed information on costs, see BDA Engineers File Note series. The reinforced brickwork retaining wall, being a more recent development does not have this large data base and as a result a detailed design and cost study has been made of the various types of brickwork retaining walls described in this note. The cost study has also compared the cost of reinforced concrete walls of the same height which either have a decorative ribbed or a fair-faced brickwork finish to be comparable in the quality of their appearance. The fair-faced structural brickwork retaining walls were designed using up-to-date current guidance as given in BS 5628:Parts 1[1], 2[2] and 3[5]. The reinforced concrete walls were designed using BS 8110[11]. Table 13 shown below is based on a maximum wall height of 6 metres and on an index system where the index of 100 represents the cheapest wall for a particular height. No figure is given for a particular type of wall when it is considered an inappropriate form of construction for that height.

For walls which are straight in planform, which are not battered back and which are of constant height, plain reinforced concrete with no decorative finish becomes the cheapest solution at heights of 3 metres or greater. Should a decorative finish be required, however, the cost equation alters: in all cases the ribbed reinforced concrete walls are more expensive than the appropriate brick solution - whether pocket, grouted cavity, mass brick or post-tensioned diaphragm wall. Indeed, the cost of the ribbed faced concrete wall is similar to a brick clad plain reinforced concrete wall for all heights

TABLE 13

Cost Index Comparison of Brickwork Retaining Walls with Reinforced Concrete Retaining Walls

WALL TYPE							
Wall Height	Grouted Cavity Wall	Mass Brickwork Wall	Post-tensioned Diaphragm Wall	Pocket Type Brickwork Wall	Plain Concrete Wall	Ribbed Finish Bush Hammered R.C. Wall	R C Wall with Brickwork Cladding
1.05	109	100		125	131	158	161
2.025	100	124		101	104	128	132
3.0	114	146		106	100	122	124
4.05 *	141		110	111	100	123	126
5.025*	171		108	115	100	121	124
6.05 *			100		102	120	122

*Cost comparison of stem of wall only. Although foundation costs vary slightly for each solution, the size of differences should not affect the cost comparison index by more than 1 or 2 points.

assuming, again, straight vertical walls of constant height. It should be remembered that comparing a plain reinforced concrete wall with a structural brickwork solution is not comparing like with like since a brickwork solution makes it a more attractive alternative in the majority of cases. Of course, should brick cladding be required, the argument to use structural brickwork becomes very strong indeed.

It is clear from this cost exercise that brickwork retaining walls are the most appropriate in terms not only of cost but of appearance, durability and maintenance during design life.

When walls are curved on plan or when the height of the wall varies along its centre line, the cost advantage of a brick solution is enhanced beyond that shown in the table below. In these circumstances, the achievable cost savings can be extremely worthwhile

6.0 ACKNOWLEDGEMENTS

The costing comparisons were prepared by J. C. Yeadon of Rex Procter and Partners, Quantity Surveyors of Leeds on behalf of Armitage Brick Limited. Their kind permission to use these costs in this publication is duly acknowledged.

7.0 REFERENCES

(1) BS5628: Part 1: 1978, **Structural use of unreinforced masonry**

(2) BS5628: Part 2: 1985 **Structural use of reinforced and prestressed masonry**

(3) CP111: 1970, **Structural recommendations for loadbearing walls**

(4) The Building Regulations 1985

(5) BS5628: Part 3: 1985, **Use of masonry: Materials and components, design and workmanship**

(6) Civil Engineering Code of Practice No. 2 (1951): **Earth Retaining Structures**

(7) Soil mechanics literature, e.g. Terzaghi & Peck, **Soil Mechanics in Engineering Practice**

(8) **External Walls: Design for Wind Loads.** Brick Development Association

(9) CP114: Part 2: 1969, **The structural use of reinforced concrete in buildings**

(10) BS3921: 1985, **Clay bricks.**

(11) BS8110: Part 1: 1985, **Structural use of concrete**

(12) BS187: Part 2: 1978, **Calcium silicate bricks**

(13) BS729: 1971, **Hot dip galvanised coatings on iron and steel articles**