Concrete
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Association

DESIGN GUIDE

FREE-STANDING WALLS

First Edition

CONCRETE MASONRY

Free-standing walls
— A Design Guide —

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PREFACE

The South African Bureau of Standards published its Code of Practice, SABS 0164 Part 1, the Structural use of Masonry : Unreinforced masonry walls, a little over a decade ago. Prior to the publication of this code of practice, the design of free-standing walls was based solely on empirical rules which imposed limitations on the heights of walls and restrictions on the type of pier employed. The range of options available to designers has been further increased by virtue of the recent publication of SABS 0164 Part 2 : Structural design and requirements for reinforced and prestressed masonry.

The demand for yard and boundary walls in excess of 1,8 m in height has grown as community needs have changed. Boundary walls erected for the purposes of security and privacy have become the order of the day. The challenge for the building industry is to provide economical, structurally sound and aesthetically pleasing boundary walls which meet the needs of the community.

This design guide provides information on the construction of unreinforced as well as reinforced walls, for use by builders, architects, engineers, developers and owners. The design guide is divided into 10 sections, viz:

1) Introduction.
2) The selection of materials.
3) Design in accordance with the National Building Regulations.
4) Accommodation of movement.
5) Rational design.
6) An approach to the routine design of unreinforced free-standing walls.
7) An approach to the routine design of reinforced free-standing walls.
8) Simplified procedures for the design of unreinforced and reinforced free-standing walls.
9) General requirements applicable to free-standing walls designed in accordance with the simplified procedures.
10) Worked examples of masonry wall design.

The requirements of the National Building Regulations in regard to free-standing walls may be satisfied by either:

- applying deemed-to-satisfy rules
- engaging the services of a competent person, usually a professional engineer, to prepare a *rational* design and to supervise the construction of such walls

Guidance is given in this design guide for the rational design of free-standing walls, both reinforced and unreinforced, in accordance with the deemed-to-satisfy structural codes of practice referred to in the Regulations. Section 5 of this design guide reviews clauses contained in these codes of practice relating to the design of free-standing walls. An approach to routine design is developed and presented in Sections 6 and 7; it culminates in the formulation of simple formulae and sets of rules by means of which walls (with or without piers) up to 3,0 m in height may be designed on a rational basis without resorting to structural calculations.

It must be emphasised, however, that the responsibility for the structural design of a specific wall rests with the appointed professional engineer. Sections 5 to 7 have been included in this guide to enable professional engineers to familiarise themselves with the relevant South African structural codes of practice and other related published work. At the same time, these sections of this Design Guide afford professional engineers the opportunity of critically examining the assumptions and the theory upon which the simplified design procedures are based, and to recognise loading conditions which fall outside the scope of these simplified procedures. Persons other than professional engineers should skip over Sections 5 to 7 and utilise Section 8 to explore possible rational solutions for specific projects.

Most builders, architects and developers have access to personal computers which operate on the MS DOS operating system. To facilitate the design of free-standing walls by means of the simplified rules contained in Section 8, a computer programme entitled FREESTAND has been written by R B Watermeyer and P I von Maltitz and is distributed by the CMA. The programme also affords engineers the opportunity to change the values of the selected variables upon which the simplified procedures are based.

In some instances, particularly where walls are exposed to high wind loads, the design of unreinforced walls predicted by means of the simplified procedures will not always be more economical than a design based on the deemed-to-satisfy rules contained in Part K of the National Building Regulations. This may be attributed to inconsistencies between deemed-to-satisfy rules and structural codes of practice.

It is hoped that, in general, this design guide will contribute to a better understanding of the structural design of free-standing walls, promote the construction of economical and structurally sound free-standing walls and reduce the design costs associated with walls designed by professional engineers.

RB Watermeyer
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1. INTRODUCTION

In SABS 0400 [1] a free-standing wall is defined as ...... *a wall not being a retaining wall, without lateral support,* whilst BS 5628 Part 3 [2] defines it as...... *a wall without top or side support that depends for stability on its mass and/or base fixity.*

Common wall forms which fall within the ambit of these definitions may be described as boundary, yard, garden, retaining, parapet and balustrade walls. Design procedures for the three most common forms of free-standing walls, namely yard, boundary and garden walls, and the construction thereof in concrete masonry, will be presented in this Design Guide. (Design procedures, design aids and further particulars relating to plain retaining, parapet and balustrade walls may be found in References 3 and 4).

2. THE SELECTION OF MATERIALS

2.1 Masonry units

The standard specification for concrete masonry units is SABS 1215 [9]. This specification covers the physical properties of units and the sampling of units for testing. Limitations are imposed by this specification on dimensional tolerances, minimum compressive strengths, drying shrinkage, expansion on rewetting and soundness.

Assurance in regard to due compliance with the requirements of this specification is obtained by way of the South African Bureau of Standards mark for the manufacture of concrete masonry units. The SABS mark may only be applied if the manufacturer is in possession of a valid permit issued by the SABS and the product complies with the requirements of SABS 1215.

2.2 Mortar constituents

2.2.1 Cementitious materials

Ordinary portland cement and Portland Cement 15 may be used in mortar. It is not advisable to use PBFC. Unless the mortar sands are of good quality (see 2.2.3), mortar with portland cement lacks cohesiveness (plasticity), may bleed and will be harsh to work. This deficiency in properties may be overcome by using masonry cement or by the addition of bedding lime or a mortar plasticizer [6].

2.2.2 Lime

The use of lime in mortar mixes is optional. Lime imparts to mortar the properties of plasticity and water retention. This latter property is important as it prevents the mortar from drying out so rapidly as to result in the incomplete hydration of the Portland Cement [6].

Lime for mortar consists of hydrated lime, i.e. commercial bedding lime, and not quicklime or agricultural lime. The best results are obtained when used in conjunction with coarse sand rather than with clayey sands. Lime complying with Class A2P of SABS 523 [22] is most suitable. Lime should not be added to masonry cement.

2.2.3 Sand

Sand for mortar should comply with SABS 1090 [23] and be well graded from a particle size of 5 mm downwards, in accordance with Table 1.

Table 1: Grading requirements of sands for mortar (Extract from SABS 1090: Sand for plaster and mortar)

B.S. Sieve (μm)	Percentage by mass passing			
	Natural sand		Manufactured sand	
	High strength mortar	General purpose mortar	High strength mortar	General purpose mortar
4 750	100	100	100	100
2 360	90 — 100	90 — 100	90 — 100	90 — 100
1 180	90 — 100	70 — 100	70 — 100	70 — 100
600	40 — 100	40 — 80	40 — 100	40 — 100
300	5 — 60	5 — 75	5 — 65	10 — 75
150	0 — 15	0 — 25	5 — 20	0 — 25
75	0 — 7,5	0 — 10	0 — 10	0 — 15

Sand should be evenly graded and should not contain an excess of dust or other fine material. The use of fine sands of more or less uniform particle size, although contributing to workability, frequently leads to excessive shrinkage and cracking of the joints. Sands containing high percentages of clay tend to give a conveniently plastic mix, but also lead to undue shrinkage [6].

In certain parts of South Africa, it is almost impossible to obtain sand from natural sources which comply with the grading requirements of SABS 1090. In such areas, the following test based on water demand, developed by the Portland Cement Institute, should be performed on sands to determine their suitability for use in the construction of free-standing walls [6].

Procedure

1. Dry out a wheelbarrow full of the sand to be tested.
2. Weigh out 5 kg cement and 25 kg of the dry sand. Measure out 5 litres, 1 litre and 1,5 litres of water into separate vessels.
3. Mix the cement and sand in a dry condition until the colour is uniform.
4. Mix in, in succession, each of the amounts of water (5 litres, 1 litre and 1,5 litres) until the mix reaches a consistency suitable for plastering.

Then:

If 5 litres is enough - the sand is of good quality.
If 5 litres + 1 litre is enough - the sand is average.
If 5 litres + 1 litre + 1,5 litres is enough - the sand is poor.
If more than 7,5 litres is needed - the sand is very poor.

Only sands which may be described as good and average should be used in free-standing walls. (When performing the abovementioned test, it is important to accurately weigh out the sand, to carry out the test on a smooth, impervious surface and to ensure that the sample is fairly representative of the bulk supply).

2.2.4 Mortar plasticizers

Mortar plasticizers produce a desirable effect on the workability or plasticity of the mortar in which they are used. Generally plasticizers have no effect on setting time (ie. accelerate or retard the mortar setting); they may, however, cause air-entrainment [6].

The use of mortar plasticizers is optional. Their effectiveness varies with the quality of sand, the composition of the cement, its fineness, the water/cement ratio, the temperature of the mortar, the amount of plasticizer used and other factors such as site conditions.

2.3 Class of mortar

Class II mortar, as defined in appendix C of SABS 0164 Part 1[8], should be used in free-standing walls. A class II mortar is described as a mortar having a field strength of at least 5 MPa at 28 days, or a mortar composed of the limiting mix proportions as set out in Table 2.

Table 2: Mix proportions for mortar [8]

Portland Cement kg	Lime litres	Sand measured loose and damp litres, max	Masonry cement or Portland cement with mortar plasticizer kg	:	Sand litres, max
50	0 — 40	200	50	:	170

2.4 Wall ties

Wire ties should comply with the requirements of SABS 28 [41]. Flat metal ties should be hot dip galvanised or protected against corrosion in accordance with the requirements of SABS 0164 Part 2 [31].

2.5 Materials for infill concrete

Portland cements complying with SABS 471[34], or cements containing ground granulated blast furnace slag or fly ash complying with the relevant requirements of SABS 626 [35] or 831 [36] or 1466 [37], may be used in infill concrete. Aggregates, both coarse (stone) and fine (sand), should comply with the requirements of SABS 1083.

2.6 **Reinforcement for masonry piers**

Steel reinforcement used in the construction of masonry walls should comply with SABS 920[38], and should be bent in accordance with SABS 82[39].

3. DESIGN IN ACCORDANCE WITH THE NATIONAL BUILDING REGULATIONS

The requirements of the National Building Regulations with respect to the design of unreinforced free-standing walls may be satisfied by either:

- applying the deemed-to-satisfy rules stipulated in parts H and K of SABS 0400; or
- engaging the services of a competent person, usually a professional engineer, to provide a rational design and to supervise the erection thereof. (See part B of SABS 0400).

Table 5 of Part K of SABS 0400 (see Table 3) tabulates the permissible heights of unreinforced free-standing walls for given nominal wall thicknesses and pier configurations. This table is based on empirical rules, some of which have been modified following the publication of a technical paper by Watermeyer [3]. Nevertheless, many of these rules remain questionable from an engineering point of view. Furthermore, no differentiation in wall height is made to compensate for varying wind conditions as is the case in BS 5628 Part 3, whilst the pier configuration is restricted to piers projecting on one side only.

Table 3: Free-standing walls (Part K, SABS 0400)

Nominal thickness mm	Maximum height above finished ground level, m		Piers	
	Without piers	With piers	Projection x width, mm	Spacing, m
90	0,8	1,2	200 x 290	1,8
110	1,0	1,4	240 x 230	1,8
140	1,3	1,6	300 x 290	2,0
190	1,5	2,0	400 x 290	2,5
230	1,8	2,3	480 x 350	3,5
290	2,2	2,6	400 x 290	4,5

Free-standing walls complying with part K should be constructed with 10,5 MPa solid units or 7,0 MPa hollow units, using Class II mortar, and not contain a damp proof course. Furthermore, such walls should be laid in any acceptable fully-bonded pattern with the cavities in piers constructed of hollow units solidly filled with concrete.

Clause KK 11.3 does, however, allow walls to consist of two or more sections as illustrated in Figure 1. The dimensions of any piers which may be required should remain constant throughout the height of the wall.

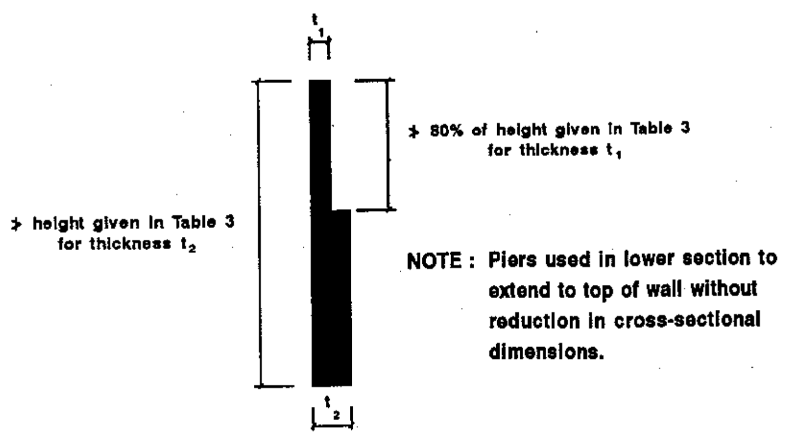


Figure 1 : Free-standing walls of composite thickness

In terms of Part H of SABS 0400, foundations for free-standing walls should:

- be constructed with concrete having a compressive strength of 10 MPa at 28 days, or be mixed in proportions by volume of 1 part of cement, 4 parts of sand and 5 parts of coarse aggregate
- have a width of 600 mm
- have a thickness of 200 mm, except in the case of rock

Foundations to piers should project by a minimum distance of 200 mm beyond the perimeter of the pier.

As an alternative, the National Building Regulations permit the preparation of a rational design in accordance with the deemed-to-satisfy codes of practice, namely SABS 0160 [5] (for loads), SABS 0161 [7] (for foundations) and SABS 0164 Part 1 [8] (for unreinforced structural masonry) or SABS 0164 Part 2 [31] (for reinforced and prestressed structural masonry). It should, however, be noted that the requirements of these Regulations do not preclude the use of other codes of practice [1].

4. ACCOMMODATION OF MOVEMENT

4.1 Minimising shrinkage in walls

Shrinkage in concrete masonry walls can be minimised by utilising units whose manufacturers hold the SABS mark and sands which either comply with SABS 1090 or may be regarded as good or average in terms of the test described in 2.2.3. Maximum bond strength and minimum shrinkage can be achieved by using concrete masonry units which are as dry as relative humidity conditions permit. The addition of moisture to the units may cause the units to expand. Upon drying, the water loss can cause shrinkage of the masonry [6].

Mortar shrinks on drying out, and the amount of drying shrinkage is directly related to the water demand of the sand. Excessive retempering of the mortar reduces mortar strength and increases shrinkage [6].

4.2 Control joints

One of the most effective ways of ensuring that free-standing walls are able to accommodate small seasonal movements due to variations in temperature and moisture is to divide the wall into discrete panels by the provision of vertical control (movement) joints, i.e. to reduce stresses by reducing restraint [6].

Table 4 summarises the magnitude of the potential movements which individual units may experience due to environmental and physical factors. In an actual wall, these movements are restrained and may therefore be expected to be somewhat smaller in magnitude than the tabulated values. As a general rule, 10 to 12 mm wide control joints should be provided at intervals of between 5 and 7m in order to accommodate the horizontal movements listed in Table 4.

Table 4: Potential movement in concrete masonry units

Movement type	Potential movement, mm/m length of wall
Thermal (reversible)	0,21 to 0,24 (for a 30° C change in temperature)
Drying shrinkage	0,60
Moisture movement (reversible)	0,12 to 0,18
Carbonation (shrinkage)	0,1 to 0,2

Differential longitudinal movements may be induced in free-standing walls founded on problem soils.These movements arise from variations in moisture content in soils which exhibit collapsible, compressible and heave/shrinkage characteristics. Vertical control joints located at the ends of walls which abut other structures, and also at a horizontal spacing of between 4 and 5 metres may, together with bed joint reinforcement, be employed to minimise damage to walls founded on such problem soil horizons [21].

5. RATIONAL DESIGN

5.1 Basic approach

In SABS 0164 Part 1, Section 5.5.4, a method of design for free-standing walls is outlined as follows:

Free-standing walls should be designed as cantilevers from the top of the foundation or from the point of horizontal lateral restraint when such restraint is sufficient to resist the horizontal reaction from the wall, except that wall panels between piers may be designed as three-sided or horizontally spanning. The piers should be designed as cantilevers to resist the reaction from the panel.

Economies in design can be effected by considering the portion of wall between piers to act as a wall panel supported on three sides. The length of any such wall panel is, however, governed by the spacing of control joints provided to accommodate movement and the strength of the supporting piers acting as cantilevers.

5.2 Notation

The following notation is adopted:

α = bending moment coefficient for wall panel

γ_f = the partial safety factor for loads

γ_m = the partial safety factor for materials

γ_{ms} = the partial safety factor for strength of steel

γ_{mv} = the partial safety factor for material strength in shear

γ_w = the density of the wall

A = the horizontal cross-sectional area of the wall

A_f = the area of a foundation

A_s = the cross sectional area of reinforcing steel

F = a coefficient which accounts for increases in wind pressures due to the geometry, the surrounding topography and the geographical location of the wall

F_1 = the wind pressure coefficient

H = height of the wall panel ie. the height of the wall above the foundation

I = the moment of inertia of a section

K = the stiffness coefficient

L = the length of a panel between supports

L^* = the overall length of the wall between discontinuities

Q_K = the characteristic imposed load

Q = the shear stress distribution coefficient

M_A = the applied moment per unit length due to ultimate loads

M_E = the elastic moment of resistance per unit length

M_d = the moment of resistance of reinforced elements

M_P = the plastic moment of resistance per unit length

M = the service moment applied to a foundation

M_{RP} = the maximum value of the ultimate moment applied to a reinforced pilaster or pier

N = the axial load applied to a foundation

R	=	the square root of the ratio of the section modulus of a wall with piers to that of a wall without piers having a thickness equal to the flange thickness of a wall with piers (elastic analysis); or the square root of the ratio of the product of the area and the lever arm of a wall with piers to that of a wall without piers having a thickness equal to the flange of the wall without piers (plastic analysis)
V	=	the horizontal shear force
V_d	=	the shear resistance of reinforced piers and pilasters
V_{RP}	=	the maximum value of the ultimate shear force applied to a reinforced pilaster or pier
V_t	=	the shear resistance of a wall tie
W_Q	=	the characteristic free stream wind velocity pressure
W_k	=	the characteristic wind load per unit area
W_R	=	the maximum wind load that a wall panel can resist
Z	=	the section modulus
Z_f	=	the section modulus of the foundation
a	=	the area above the shear plane under consideration
b^*	=	the section width at the shear plane under consideration
b	=	the width of the section
d	=	the effective depth
f	=	foundation pressure
f_{kx}	=	the characteristic flexural strength at the critical section
f_k	=	the characteristic compressive strength
f_v	=	the characteristic horizontal shear strength
f_{kv}	=	the characteristic vertical shear strength
f_y	=	the characteristic tensile strength of steel
g_A	=	the design vertical load per unit area
h_L	=	the vertical distance between the point of application of the horizontal load and the top of the foundation
h	=	the clear height of the wall above finished ground level
k	=	the ratio of overall pier depth to wall thickness
m	=	the ratio of pier width to wall thickness
n	=	the ratio of total effective flange width to wall thickness
n^*	=	the number of wall ties provided per unit length of wall
n_w	=	the design vertical load per unit length of wall
p	=	the ratio of the cross sectional area of primary reinforcement to the cross sectional area of the reinforced element
p^*	=	the proportion of the shear plane that is intersected by masonry headers
r	=	the width of a rectangular wall tie
s	=	the pier spacing
t	=	the wall or flange thickness in unreinforced walls and the wall panel thickness in the case of reinforced walls
t_{ef}	=	the effective wall thickness

t_{eq} = the equivalent wall thickness

t_s = the thickness of a rectangular wall tie

v_h = the average shear stress

v^* = the local shear stress at the section under consideration

w_f = the effective flange width of a beam

x = the vertical distance between the top of the foundation and finished ground level

y = the distance from the neutral axis to the centroid of the area above the plane under consideration

z = lever arm

5.3 Bending moments and shear forces

5.3.1 Unreinforced walls

The following formulae are given in SABS 0164 Part 1 for the calculation of the design moment and the moment of resistance of a free-standing wall based on two design approaches, viz. flexural resistance across uncracked bed joints (elastic analysis), and cracked sections (plastic analysis):

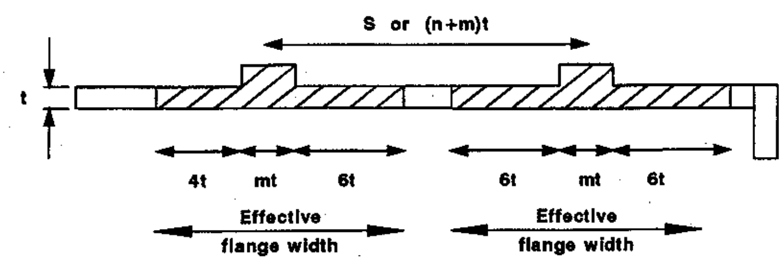
Design moment $$M_A = W_K\,\gamma_f\,h\left(\frac{h}{2}+x\right)+Q_K\,\gamma_f\,h_L \qquad \text{.............(1)}$$

Resistance moment (uncracked) $$M_E = \left(\frac{f_{kx}}{\gamma_m}+g_A\right)Z \qquad \text{.............(2)}$$

Resistance moment (cracked) $$M_P = \frac{n_w}{2}\left(t-\frac{n_w\,\gamma_m}{f_k}\right) \qquad \text{.............(3)}$$

It should be appreciated that the design approach based on equation (3) recognises that the wall may in fact crack, and probably rock, prior to failure by overturning [25]. Where the plastic (cracked) moment of resistance exceeds that of the elastic (uncracked)section, cracking of the horizontal bed joint may occur. Depending on the client's requirements and whether or not the wall is plastered, this cracking may or may not represent a serviceability failure [3]. Consequently the design moment of resistance is usually based on the elastic approach (Equation 2).

SABS 0164 recommends that the section modulus of walls with piers be based on flange widths measured from the face of the pier, equal to either 4 x the flange thickness where the wall is unrestrained or 6 x the flange thickness where the wall is restrained or continuous (see Figure 2).



NOTE : Effective flange width $\not>$ S/2

Figure 2 : Effective flange widths

Free-standing walls usually comprise unreinforced wall panels supported by unreinforced / reinforced cantilever piers. Unreinforced wall panels are readily designed using the bending moment coefficients (α) contained in Table 10 of SABS 0164 Part 1 which are based on yield line theory [18, 32] i.e.:

Design moment $M_A = \alpha W_k \Upsilon_f L^2$ ....(4)

Resistance moment (uncracked) $M_E = \frac{f_{kx}}{\Upsilon_m} Z$ ....(5)

(M_E is based on the value of f_{kx} where the failure plane is perpendicular to the bed joints)

Yield line theory may be used to derive bending moment coefficients for panels supported along two adjacent edges.

In terms of shear, the sole requirement of the Code is that the following relationship be satisfied:

$$v_h < \frac{f_v}{\Upsilon_{mv}} \qquad \text{............(6)}$$

where

$$v_h = \frac{\Upsilon_f V}{A} \qquad \text{............(7)}$$

SABS 0164 Part 1 gives Υ_{mv} a value of 2,5 and f_v a value of 0,35 MPa and 0,15 MPa for Class I and Class II mortar, respectively. However, the strength of Class II mortar is superior to that of a Designation (ii) mortar as defined in BS 5628 Part 1. It is, therefore, recommended that the value for f_v given in BS 5628 Part 1, namely 0,35 MPa, be adopted in respect of a Class II mortar.

The horizontal shear stress, except at the damp proof course [19], may be increased by 0,6 times the vertical dead load per unit area of the wall cross section subject to f_v not exceeding 1,75 MPa. A draft amendment to BS 5628 Part 1 [28] recommends that the value of f_v for brickwork in the vertical direction be increased to 0,7 MPa.

SABS 0164 Part 1 is based on the British Code of Practice BS 5628 Part 1 [17]. The handbook to BS 5628 Part 1 [25] states that equation (6) is based on the assumption that walls have a simple rectangular plan and that the stress distribution is uniform across the section, but recommends that if it is more appropriate to assume an alternative distribution, it will be necessary to calculate the maximum shear stress and to compare this value to the design shear stress.

The American ACI-ASCE Building Code ACI 530/ASCE 5 [29] contains the following equation to determine the shear distribution across a wall:

$$v^* = \frac{\Upsilon_f\ Vay}{Ib^*} \qquad \text{............ (8)}$$

Equation (8), in the case of a rectangular section, may be expressed as:

$$v^* = \frac{\Upsilon_f\ 1,5V}{bt} \qquad \text{............(9)}$$

Equation (8) may also be used to evaluate the shear stress developed in the planes of the interfaces between leaves in a multileaf wall, or between a rib or a pier and the flange of the wall. It is important, particularly where collar-joints (vertical mortared or grouted joints between leaves, or a leaf and a pier/rib) are provided, to ensure that the vertical shear resistance of a wall section is adequate in order to verify that composite action of the wall in question will, in fact, take place.

SABS 0164 Part 1 offers no design information in this regard and simply permits double-leaf walls to be designed as single-leaf walls, provided that the collar joint is solidly filled with mortar or grout as the work proceeds and that 20 x 3 mm flat metal ties are provided at vertical and horizontal intervals not exceeding 450 mm. This requirement is rather onerous and somewhat impractical as it implies that the leaves, on either side of the tie, should be raised simultaneously. Watermeyer [18] conducted laboratory measurements on the shear resistance of 36 solidly filled collar-jointed clay brick wallettes, with and without metal ties, in order to evaluate the shear resistance of collar-jointed walls and the influence of construction procedures and types of ties. The results of this study are presented in Table 5.

Table 5: The shear strength of collar joints [18]

Method of construction	Average shear stress MPa	Standard deviation MPa	Characteristic shear stress MPa
Leaves built up separately — no ties	0,42	0,09	0,24
Leaves built up simultaneously — no ties — 20 x 4 mm flat metal ties — 3,5 mm dia crimp wire ties	 0,53 0,55 0,51	 0,12 0,09 0,12	 0,30 0,39 0,29

Values given for f_{kv} in other codes of practice are contained in Table 6, whilst Table 7 gives the recommended spacing of headers and ties in double leaf walls where monolithic (composite) structural action is required. Where ties are used, the collar joints must be solidly filled with mortar and grout and should not exceed 25 mm in width. In the case of interfaces at ribs and piers, AS 3700 recommends that headers be provided every fourth course, or alternatively one tie be provided for every 200 mm width at a vertical spacing not exceeding 400 mm.

Table 6: Values for characteristic vertical shear strength [19,28,29]

Code of Practice	f_{kv}, MPa	Remarks
AS 3700	1,25 f_{kx} 1,20 p* +	Wire ties provided; $f_{kv} \leq 0{,}35$ MPa Masonry headers provided; $f_{kv} \leq 0{,}6$ MPa
BS 5628 Part 1 (Draft Amendment)	0,7	Brick masonry
ACI 530/ASCE 5	0,036 *§ 0,073 *§ 0,085 $\sqrt{f_k}$ §	Mortared collar-joints with wire ties Grouted collar-joints with wire ties Headers

+ p* is the proportion of shear plane that is intersected by masonry headers
* Values can range from 0,036 to 0,073 MPa depending on the type and condition of the interface, the consolidation of the joint and the type of loading [30]
§ Working stress value, which incorporates factors of safety

Table 7: Spacing of headers and ties in double leaf walls [8,17,29,29]

Code of Practice	Tie requirements	Header requirements
SABS 0164 Part 1 and BS 5628 Part 1	Flat metal (20 x 3 mm) ties at not more than 450 mm horizontal and vertical intervals.	One complete header course to every 5 or less stretcher courses, or the equivalent area of evenly distributed headers.
AS 3700	Cavity type ties at intervals not exceeding 400 mm centres in each direction	Header course with each alternate unit a header at intervals not exceeding 600 mm or the equivalent number of evenly distributed header units.
ACI 530/ASCE 5	2,4 to 4 ties per square metre depending on the diameter of the tie, subject to a maximum horizontal spacing of 900 mm and a vertical spacing of 600 mm.	Headers uniformly distributed so that the sum of their cross sectional areas is greater than 4% of the wall area.

In a draft amendment to BS 5628 Part 1, it is recommended that the vertical shear stress between two elements of a section, such as at the junction between the flange and rib of a diaphragm wall, should be resisted by either bonded masonry construction bridging the interface or appropriate flat metal sections, the size and spacing of which should be calculated. Roberts [20] has developed the following formula to determine the shear capacity of wall ties:

$$V_t = \frac{f_y \, r \, t_s}{6 \, \Upsilon_m} \qquad \text{........... (10)}$$

where $\Upsilon_m = 3{,}0$

Alternatively, the values for the shear resistance of ties contained in Table 9 of SABS 0164 may be utilised.

5.3.2 Reinforced walls

SABS 0164 Part 2 contains the following formulae for the evaluation of the design moment of resistance of reinforced members:

Resistance moment $$M_d = \frac{A_s f_y z}{\Upsilon_{ms}} \qquad \text{.............(11)}$$

where $$z = d\left(1 - \frac{0{,}5\, A_s\, f_y\, \Upsilon_m}{bd\, f_k\, \Upsilon_{ms}}\right)$$

provided, that $$M_d \leq \frac{0{,}4\, f_k\, bd^2}{\Upsilon_m} \qquad \text{.............(12)}$$

Equation (12) may not apply to members comprising pilaster units, as the determination of the value for f_k of such units is beyond the scope of SABS 0164. Should the pilaster units be treated as shutters and minor cracks within the pilaster itself be permitted, such members may be designed as reinforced concrete members in accordance with a Code of Practice such as SABS 0100.[40]

Equation (7) may be used to evaluate the applied shear stress. The magnitude of the shear resistance of a reinforced pier is given by:

$$f_v = 0{,}35 + 17{,}5p \qquad \text{.............(13)}$$

provided that $f_v \leq 0{,}7$ MPa.

5.4 Basic unit properties

In SABS 0164 Part 1, the size of a brick and of a block is not defined; it is, however, based on British practice according to which, until fairly recently, a brick was regarded as a masonry unit not exceeding 337,5 mm in length, 225 mm in thickness (width) and 112,5 mm in height. (Currently, British codes refer only to masonry units.) Consequently, the adoption of the aforementioned dimensions rounded to 340, 225 and 115, respectively, is considered to be appropriate for the purpose of interpretating the characteristic flexural tensile stresses in respect of bricks and blocks, as tabulated in SABS 0164 [4].

The basic structural properties of common concrete masonry units having nominal compressive strengths of 10,5 and 7,0 MPa for solid and hollow units, respectively, are summarised in Table 8 [8,16].

Table 8: Structural properties of concrete masonry units

Unit type	Unit width mm	Characteristic flexural tensile strength f_{kx}, MPa		Characteristic compressive strength f_k, MPa	Density of masonry Υ_w, kN/m³	
		Parallel to bed joints	Perpendicular to bed joints		Natural Aggregate	Clinker Aggregate
solid+	90 — 140	0,30	0,90	4,1 — 4,8	22,0	18,0
hollow*	90	0,25	0,60	5,1	15,4	12,6
hollow*	140	0,22	0,53	4,2	12,1	9,9
hollow*	190	0,19	0,45	3,7	11,2	9,2

\+ Height not exceeding 115 mm
* Height = 190 mm

5.5 Partial safety factors

5.5.1 Loads (Υ_f)

SABS 0164 Part 1 lays down partial safety factors of 0,9, 1,4 and 1,6 for minimum self weight loads, wind loads, and imposed loads, respectively. However, in the case of free-standing walls, the value of 1,4 for wind loads may be reduced to 1,2. This is consistent with the values given for wind loads in SABS 0160 if the consequence of failure is taken into account, ie. 0,9 x 1,3 = 1,17, say 1,2.

Similarly, on the premise that free-standing walls present a low degree of hazard to life and limb, the partial safety factor for imposed loads may be considered to be 0,9 x 1,6 = 1,44 say 1,45.

Thus the following partial safety factors should be adopted in the design of free-standing walls:

- 0,9 for dead loads contributing to stability or flexural resistance
- 1,2 for wind loads
- 1,45 for imposed loads

SABS 0164 Part 2, on the other hand, simply adopts the partial safety factors laid down in SABS 0160 which are as described above.

5.5.2 Materials (Υ_m)

SABS 0164 Part 1 lays down partial safety factors of 2,9; 3,2 and 3,5 for the compressive and flexural strengths of masonry, depending on the manufacturing control and construction control exercised. The cost of testing of both the mortar mix designs and site batching so as to satisfy the higher levels of construction control is not usually warranted by savings attainable in the design of free-standing walls. However, the benefits derived by using masonry units supplied by manufacturers who hold the SABS mark for their products and thus qualify for the higher level of manufacturing control, are considered to be highly advantageous.

Where units are supplied by manufacturers who hold the SABS mark, a partial safety factor of 3,2 may be employed, failing which a factor of 3,5 should be applied.

The tabulated values of partial safety factors as stated in SABS Part 2, on the other hand, are based on the assumption that construction control will be duly exercised. Considering that the reinforcement in reinforced free-standing walls of a height not exceeding that contemplated in this design guide is placed only within the piers, it is recommended that construction control need not be exercised, and the aforementioned partial safety factors as described for unreinforced masonry be adopted. (SABS 0164 Part 2 recommends the use of higher safety factors where construction control is not exercised.)

Values of 2,5 and 3,0, as laid down in Part 1, should be used as the partial safety factor for shear and wall ties, respectively. Similarly, in the case of reinforced masonry, values of 2,0; 1,4; and 1,15 as laid down in Part 2, should be adopted for shear, bond and steel strength, respectively.

5.6 Design loads

5.6.1 Code requirements

SABS 0160 [5] requires that all walls be designed to resist the appropriate wind forces. However, if a wall exceeds 1,5 m in height, the following imposed horizontal loads, acting independently from wind loads, should be resisted:

- a concentrated force of 1,0 kN acting at a height of 1,8 m or at the top of the wall, whichever is the lesser
- a uniformly distributed force of 0,36 kN/m acting at a height of 1,1 m

In masonry, concentrated forces may be assumed to be distributed at an angle of 45° towards the support. Consequently, stresses induced by concentrated loads may be expected to be low in magnitude, except in the case of short walls.

5.6.2 Free-stream velocity pressures

South Africa can be divided into distinct wind velocity pressure zones based on the product of the altitude correction factor and the square of the characteristic wind speed. Figure 3 shows the division of the country into four such zones [10].

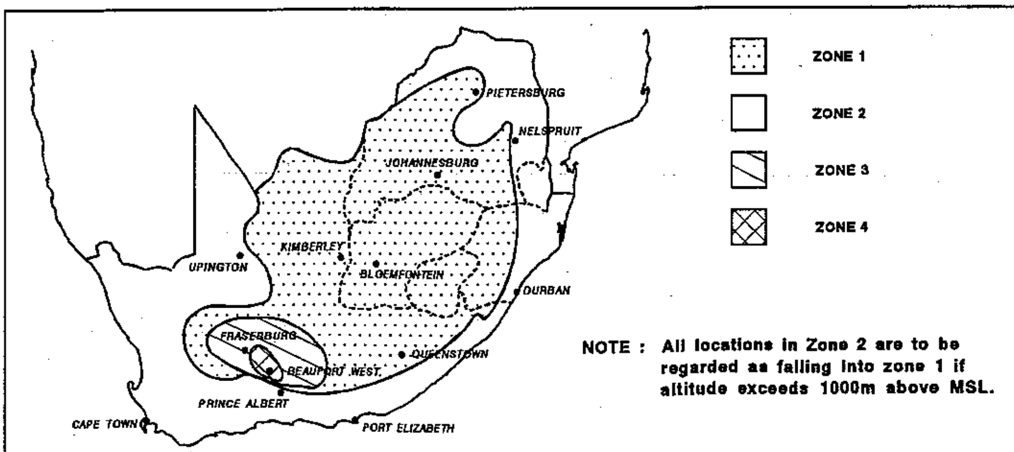


Figure 3 : Wind velocity pressure zones

Wind speeds are usually measured at 10 m above ground level. SABS 0160 provides terrain height multipliers for four terrain categories, as described in Table 9, to enable wind speeds to be determined at heights ranging from 5 to 500 m above ground level.

Table 9: Terrain Categories

Category	Description	Examples
1	Virtually no obstructions. Obstruction height less than 1,5 m.	Open sea coasts; lake shores; flat treeless plains.
2	Widely spaced obstructions (>100 m apart) with heights and plan dimensions generally between 1,5 and 10 m.	Airfields; open parkland; undeveloped out–skirts of townships.
3	Closely spaced obstructions generally the size of a domestic house.	Wooded areas; suburbs; towns; industrial areas.
4	Numerous large, tall (10 to 30 m high), closely-spaced obstructions.	Large city centres.

The values of the terrain/height multipliers tabulated for the lower range of heights are based on the assumption that below heights of 5; 5; 7,5 and 20 m in terrain categories 1, 2, 3 and 4, respectively, the wind will be strongly influenced by local obstructions which may cause acceleration of the flow in their wake [5]. SABS 0160 does not, therefore, permit reductions in wind speed at heights below the aforementioned values.

This approach is a departure from earlier British codes of practice such as CP3 : Chapter V [15] where arbitrary values were assigned for heights ranging from 3 to 10 m [11] to accommodate the effect of shielding by upwind buildings. The most recent Australian code of practice, AS 1170.2-1989 [14], tabulates terrain/height multipliers at a height of 3 m only in terrain categories 1 and 2, and introduces an additional wind speed multiplier to account for shielding effects under certain conditions in terrain categories 3 and 4.

Free-stream velocity pressures for a 25 year period, applicable to free-standing walls up to 3 m high, for the wind zones and terrain categories defined in Figure 3 and Table 9, respectively, are tabulated in Table 10. In terrain categories 1 and 2, the terrain/height multipliers used in the derivation of the tabulated values have been adjusted to a height of 3 m above ground level so as to reflect Australian practice. In terrain categories 3 and 4, the tabulated values may be reduced by a shielding factor, provided that upwind buildings within a 45° sector of radius 20h are greater than or equal to h [14]; full particulars in this regard are contained in Appendix A.

Table 10: Free stream velocity pressures (W_Q, kN/m^2)

Zone	Terrain category			
	1	2	3	4
1	0,73	0,58	0,32	0,30
2	0,83	0,66	0,37	0,35
3	0,96	0,77	0,43	0,40
4	1,18	0,94	0,52	0,49

The values for free stream velocity pressures tabulated in Table 10 need to be modified to account for local topographical features such as funnelling effects and sudden changes in ground conditions, i.e. at crests of hills, ridges and escarpments. Guidance for such modifications is given in SABS 0160 and AS 1170.2. (AS 1170.2 modifies free-stream velocity pressures at crests of hills, ridges and escarpments only if the upwind slope, measured as the ratio of the height of the hill, ridge or escarpment to twice the horizontal distance from the crest to a level half the height below the crest, exceeds 0,05 - see Figure 4).

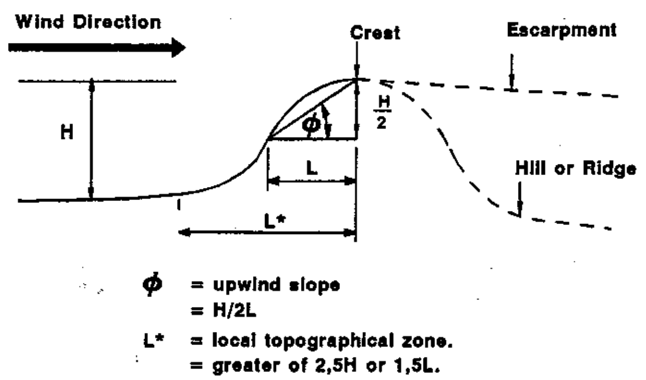


Figure 4 : Hills, ridges and escarpments

The wind speed profile for a given terrain category does not develop to its full height at the commencement of that terrain category, but is gradually established. Particular care must be taken in assessing the applicable terrain category where walls are located in residential townships in close proximity to dissimilar terrain categories, e.g. large open spaces. Guidance is given in SABS 0160 in regard to the determination of the fetch distance (the distance in an upwind direction to the wall, from the edge of the terrain category) required before the fully developed free-stream velocity pressure for the new terrain category can be applied.

5.6.3 Pressure coefficients

SABS 0160 tabulates pressure coefficients for a range of low walls with varying width to height ratios in respect of wind perpendicular to the wall. These coefficients are derived from tables contained in the British Code of Practice, CP3: Chapter V[15], for unclad structures[11].

However, wind tunnel tests undertaken during the mid-1980s on free-standing walls, both in Australia and the United Kingdom, indicated that the maximum net pressures occurred at the windward end of the wall near a free end or corner when the wind direction was at approximately 45° to the wall[12]. (The common mode of failure is one where the end region fractures and falls, leaving the remainder of the wall standing with, typically, a 45° taper to the ground). This phenomenon was found to be more pronounced as the length of a wall increased. The presence of an adjacent wall, at right angles to a free end was found to significantly reduce the net wind load near the corner[12,13].

It is, therefore, considered appropriate to adopt the force coefficients contained in the British draft code of practice BS 6399[27] and the Australian code of practice AS 1170.2 for the design of free-standing walls. These coefficients are reproduced in Tables 11 and 12, respectively, and are graphically illustrated in Figure 5.

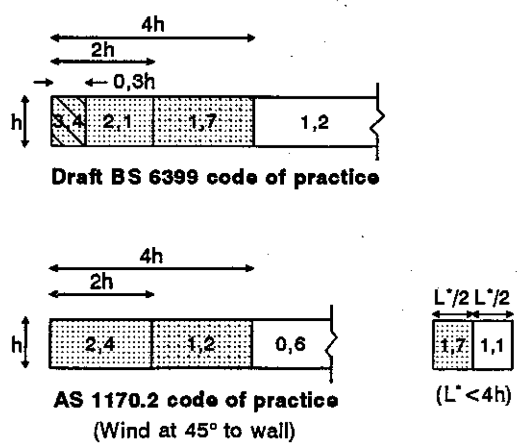


Figure 5 : Force coefficients for free-standing walls without end returns

Table 11: Force coefficients for free-standing walls (Draft BS 6399)

Distance from windward end (see Figure 5)	Force coefficient	
	Wall without end returns	Wall with end returns
0 to 0,3h	3,4	2,1
0,3h to 2h	2,1	1,8
2h to 4h	1,7	1,4
≥4h	1,2	1,2

Table 12: Force coefficients for free-standing walls ($0,5 \leq L^*/h \leq 45$) (AS 1170.2)

Distance from windward free end (see Figure 5)		Wind direction	Force coefficient
—		Normal	1,2
$L^* > 4h$	0 to 2h	45°	2,4
	2h to 4h	45°	1,2
	> 4h	45°	0,6
$L^* < 4h$	0 to $L^*/2$	45°	1,7
	$L^*/2$ to L^*	45°	1,1

5.7 **Foundation design**

Foundations should be designed so as to transmit horizontal forces and overturning moments into the ground without causing the wall to slide, to overturn or to load the subsoil beyond its safe bearing capacity.

SABS 0161 [7] requires that a plain concrete foundation be proportioned so that either the ratio of its projection beyond the face of a wall, or pier, to its vertical thickness does not exceed 1,5; or the maximum tensile stress in the foundation concrete at the face of the wall or pier does not exceed 0,03 times the 28 day compressive strength of the concrete.

5.8 **Limiting dimensions**

SABS 0164 Part 1 does not limit deflections in walls by way of deflection ratios or other criteria, as is the case in codes of practice for other materials. However, the code does require that laterally loaded walls satisfy limiting dimensions, which are akin to slenderness ratios for vertically loaded walls, in order to ensure that walls are not sensitive to errors of design and construction, and to prevent instability [25]. In the case of free-standing walls these limits are:

$$h \leq 12\, t_{ef} \qquad \text{...........(14)}$$

where t_{ef} is equal to the actual thickness of walls without piers, or the flange thickness of a wall with piers, multiplied by the coefficient K given in Table 13.

Table 13: Stiffness coefficient for wall stiffened by piers.

Ratio $\frac{s}{mt}$	Stiffness coefficient, K	
	k = 2	k ≥ 3
6	1,4	2,0
8	1,3	1,7
10	1,2	1,4
15	1,1	1,2
20 or more	1,0	1,0

(AS 3700 uses the term *design for robustness* as opposed to *limiting dimensions*, and suggests that, in diaphragm construction, t_{ef} should be taken as the overall thickness of the wall. The handbook to BS 5628 [25] recommends that, in the case of Z-shaped walls, t_{ef} should be taken as the overall thickness of the wall.)

Limiting dimensions are also given in respect of wall panels. SABS 0164 requires that the area of panels which are supported on three sides may not exceed $1350t^2$ or $1500t^2$, depending upon the degree of fixity of the supports, whilst the length of such panels between supports may not exceed 50t. AS 3700, in the case of wall panels supported on two sides, recommends that the horizontal dimension should not exceed 14t. This value is, however, thought to be somewhat conservative, and a value of 20t is considered to be more appropriate.

SABS 0164 Part 2, in the case of reinforced cantilever beam elements, requires that the height to effective depth ratio not exceed 7. However, since such elements are free-standing and are subjected predominantly to wind loads, it is suggested that, by the same reasoning that a 20% increase is permitted in the case of free-standing cantilever walls, this value be increased to 8,4.

The handbook to BS 5628 Part 2 [33] considers these ratios to be somewhat conservative. AS 3700, in the case of isolated reinforced piers, recommends that, for robustness, isolated piers should have a height not exceeding 9 times the minimum pier thickness.

5.9 Detailing of reinforced masonry

SABS 0164 Part 2 contains guidance on the detailing of unreinforced masonry; the main guidelines of which are summarised below.

5.9.1 The minimum horizontal distance between parallel bars should not be less than the bar diameter, the maximum size of aggregate plus 5 mm, or 10 mm, which ever is the greater.

5.9.2 Only one reinforcing bar should be used in vertical pockets or cores of cross section not exceeding 125 mm x 125 mm.

5.9.3 The lap length should not be less than 25 times the bar size plus 150 mm or the applicable anchorage bond length. (A fully stressed hot rolled high yield deformed bar requires an anchorage bond length of 68 times the bar size when placed in mortar, and 55 times the bar size when placed in infill concrete.)

5.9.4 The effective anchorage length of a hook or bend, measured from the start of the bend to a point four times the bar size beyond the end of the bend, should be taken as the lesser of: 24 times the bar size; or eight times the internal radius of a hook; or four times the radius of the bend; as appropriate.

5.9.5 Reinforcing bars, located in cores in hollow units and piers where concrete infill is employed, require no corrosion protection unless the member in which they are placed comes into contact with aggressive soils, sea water or corrosive liquids and gases. Where mortar infill is employed, bars should, in industrial environments or where members come into contact with fresh water or are located in coastal regions, be galvanised or coated.

5.9.6 The cover to reinforcing bars located in cores in hollow units and piers filled with concrete, is dependent on exposure conditions, and on the grade and the minimum cement contents of the concrete.

6. AN APPROACH TO THE ROUTINE DESIGN OF UNREINFORCED FREE-STANDING WALLS

6.1 Basic assumptions

The routine design of a wide range of walls can be facilitated by the use of design charts [3,4]. In the derivation of the design aids and charts which follow, the following assumptions are made:

- No damp-proof courses are present in walls
- Class II mortar is used (see Section 2.3)
- Sands either comply with SABS 1090 or may be described as *good or average* in terms of the PCI test (see Section 2.2.3)
- The minimum nominal compressive strengths of concrete masonry solid and hollow units are 10,5 MPa and 7,0 MPa ,respectively
- Concrete masonry units, in addition to complying with SABS 1215, have the SABS mark (see Section 5.5.2)
- The minimum densities of concrete masonry units are as listed in Table 8
- Piers are adequately bonded or tied to walls (see Section 5.3)
- All collar-joints in multi-leaf walls are solidly filled with mortar and are capable of transmitting shear across the collar-joint (see Section 5.3)
- Walls do not retain soil or other materials exerting horizontal loading
- Walls are not located in places of public assembly
- The cavities in hollow blocks forming piers and ribs are solidly filled with mortar, grout or infill concrete
- Walls either terminate in piers or are fully bonded to returns

6.2 **Bending moments**

It can be demonstrated that the ultimate moment due to uniformly distributed line loads acting on the walls is always less than or equal to the ultimate moment due to wind loads. Consequently, all horizontal loading except wind loading is ignored in the structural design of walls.

Design calculations for walls without piers may be facilitated by equating the expressions for M_A and M_E in Equations (1) and (2), and the expressions M_A and M_P in Equations (1) and (3), and solving for t. The equations generated for walls without piers are as follows:

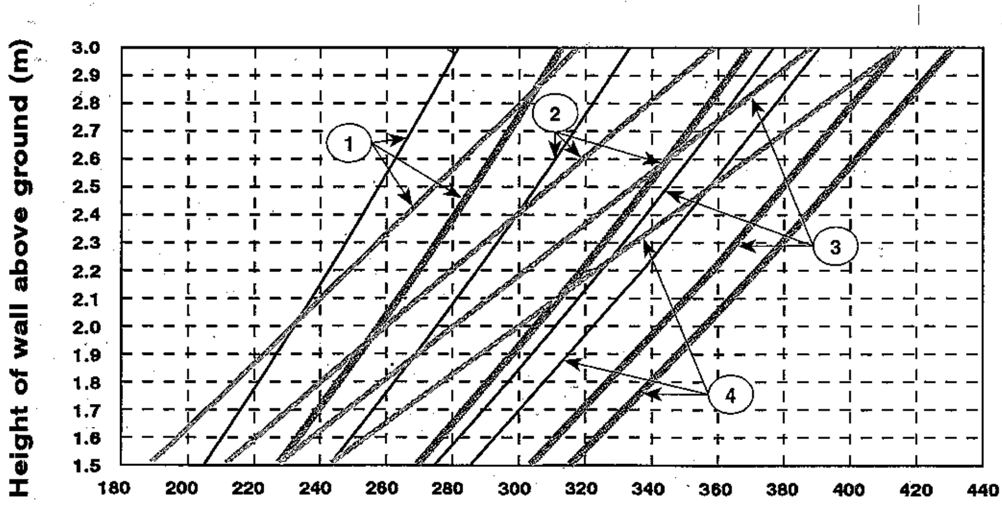
- Uncracked walls:

$$t = \sqrt{\frac{W_k\ \Upsilon_f\ h\,(\frac{h}{2}+x)}{[\frac{f_{kx}}{\Upsilon_m} + 0{,}9\,\Upsilon_w\ 10^{-3}\,(h+x)]\ \frac{1}{6}}} \qquad \text{.............(15)}$$

- Cracked walls:

$$t = \sqrt{\frac{W_k\ \Upsilon_f\ h\,(\frac{h}{2}+x)}{\frac{0{,}9\,\Upsilon_w\,(h+x)}{2}\ [1-\frac{0{,}9\,\Upsilon_w\,(h+x)\Upsilon_m}{f_k}]}} \qquad \text{............(16)}$$

It should be noted in Equations (15) and (16) that t is proportional to the square root of the wind load. By fixing x and assuming a specific value for wind loading, design graphs can be produced to enable the height of a wall to be determined for any given wall thickness. Such a design chart is presented in Figure 6 which is based on values of d = 0,3 m and wind loading = 1,2 x 0,32. The maximum permissible height of a wall for other values of W_k may be determined by adopting a wall thickness equal to the product of the actual wall thickness and the square root of the ratio of the desired wind loading W_k to the value for wind loading used in the determination of the charts (F).



LEGEND

CRACKED SECTION (PLASTIC ANALYSIS)
clinker aggregates
natural aggregates

UNCRACKED SECTION (ELASTIC ANALYSIS)
all aggregates

① 90 - 140 mm solid units
② 90 mm hollow units
③ 140 mm hollow units
④ 190 mm hollow units

Note:

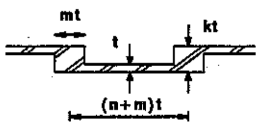
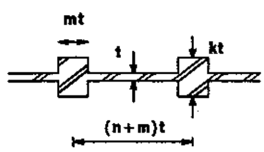
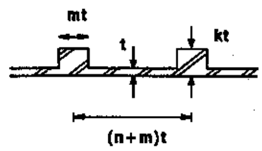
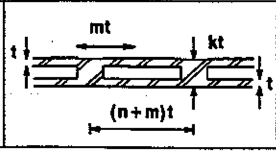
Design charts are based on Equations (15) and (16) where:
d=0,3 m
W_k = 1,2 x 0,320 kN/m^2
γ_m =3,2 (ie units bear the SABS mark)
f_k, f_{kx} and γ_w have the values given in Table 8

Figure 6 : Height of free-standing concrete masonry walls subjected to wind loads

The elastic section modulus (Z) of walls with different pier configurations can be expressed as a function proportional to n, m, k and t. As Z is directly proportional to the square of the wall thickness, the ratio (R^2) of the section modulus of a wall with piers to that of a wall without piers, of a thickness equal to the flange thickness of the former wall, is independent of wall thickness (see Figure 7). If the right hand side of equation (15) is multiplied by R, t will represent the flange thickness of a wall with piers of a given configuration. It is, however, more convenient to multiply the flange thickness by R, and to consider the product of these two parameters to be equivalent to the thickness of a wall without piers.This *equivalent wall* without piers will then have the same section modulus as the wall with the given pier configuration, and the maximum height of such a wall may be evaluated directly from Figure 6.

It is worthy of note that SABS 0164 does not differentiate between hollow and solid blocks when tabulating characteristic flexural strengths. The curves and formulae reproduced in Figures 6 and 7 are based on solid sections. Theoretically, the section modulus of single-leaf hollow block walls constructed of 90, 140 and 190 mm wide units should have been reduced by 5, 14 and 19%, respectively. Roberts et al [26] suggest that the characteristic flexural strength of hollow blocks tabulated in BS 5628 Part 1 could be based on their nett strength as opposed to the gross strength, which would have the effect of increasing f_{kx}. In the case of 140 and 190 mm wide blocks, f_{kx} would increase to 0,25 MPa i.e. an increase of between 14 and 32%. On the other hand, f_{kx} for dense concrete hollow blocks appears to be undervalued in SABS 0164 as well as in BS 5628 Part 1. It is, therefore, considered appropriate to ignore reductions in section properties due to cores in blocks.

The value of n, in most codes of practice, is usually given as 12 [8, 17, 19 and 29]. This value is based on empirical considerations [30]. In the case of walls designed on the basis of a plastic (gravity stability) analysis, it may be argued that n should be increased on the grounds that the portion of wall between piers contributes to the resistance moment of the wall. This may be true if the wall between piers is infinitely rigid, both in and out of the plane of the wall, and the wall rotates as a rigid body in response to lateral loading. In practice, however, the portion of wall between piers will deflect both in and out of the plane of the wall, and will tend to drag between piers. It is therefore considered appropriate to restrict the flange width to (12 + m)t, and to ignore the contribution to stability made by segments of walls outside these limits.

Description	Pier Configuration	Factor for Uncracked Section (R)	Factor for Cracked Section (R)
Z-shaped		$\sqrt{\frac{3n(k-1)^2 + mk^3 + n}{k(n+m)}}$	$\sqrt{\frac{n+mk}{n+m}\left[k - 2c\frac{n+mk}{n+m}\right]}$
Piers projecting equally on both sides		$\sqrt{\frac{n + mk^3}{k(n+m)}}$	$\sqrt{\frac{n+mk}{n+m}\left[k - \frac{n+mk}{m}c\right]}$
Piers projecting on one side only		$2{,}45\sqrt{\frac{\frac{n}{12}+\frac{mk^3}{12}+n(A-\frac{1}{2})^2+km(A-\frac{k}{2})^2}{A(n+m)}}$ OR $2{,}45\sqrt{\frac{\frac{n}{12}+\frac{mk^3}{12}+n(A-\frac{1}{2})^2+km(A-\frac{k}{2})^2}{(n+m)(k-A)}}$	$\sqrt{\frac{n+mk}{n+m}\left[\frac{n+mk^2}{n+mk} - \frac{n+mk}{n+m}c\right]}$ OR $\sqrt{2\frac{n+mk}{n+m}\left[k - \frac{n+mk^2}{2(n+mk)} - \frac{n+mk}{mc}\right]}$
Diaphragm		$\sqrt{\frac{(n+m)k^3 - n(k-2)^3}{(n+m)k}}$	$\sqrt{\frac{2n+mk}{n+m}\left[k - c\frac{2n+mk}{n+m}\right]}$

NOTE : $c = \frac{0{,}9\,\gamma_w(h+d)\,\gamma_m}{f_k}$ **AND** $A = \frac{n + mk^2}{2n + 2mk}$

Figure 7 : Equivalent wall thickness factors

The equivalent wall concept outlined above can be extended to include walls with piers designed on the basis of a cracked section. This can be achieved by considering R to be square root of the ratio of the product of the area and the lever arm, adjusted to prevent local crushing, for a wall with piers to that of a wall without piers of a thickness equal to the flange of the wall with piers. However, the resulting equation for R is not independent of d, h, f_k and Υ_w. By selecting values for these parameters to produce the maximum reduction in lever arm for the range of wall heights covered in Figure 6, a conservative estimate of R may be made. R will then be related to k, m, n and t, as is the case in the elastic analysis (see Figure 7).

If a pier spacing of (12 + m)t or less is adopted (i.e. n = 12), the wall with piers will act as a series of contiguous cantilever beams since the pier spacing equals the effective flange width of the beam (w_f). If piers are spaced at intervals in excess of (12+m)t, the behaviour of the wall will be similar to that of a series of cantilevered beams supporting wall panels which span between such beams. The influence of the increase in load shed by the panels onto the cantilever beams may be assessed on the assumption that the panel load disperses at 45° with respect to the supports. This may be regarded as an increase in wind loading and may be incorporated into the ratio F. At the same time, the effective width of the cantilever beam needs to be introduced into the equation since $R\,t^2 / 6$ equals the section modulus per unit width. (It should be noted that even if $w_f = (12 + m)\,t$, panel action between the centre lines of the piers may still occur.)

Walls with or without piers may be designed on the basis of either a cracked or uncracked section with the aid of Figure 6 by simply ensuring that the following relationship is satisfied:

$$t R \sqrt{w_f} \leq FX \qquad \text{.............(17)}$$

where X is the equivalent wall thickness, i.e. the ordinate in Figure 6. In practice, it is more convenient to express equation (17) as:

$$t_{eq} = f_1 \; f_2 \; f_3 \; f_4 \; t \; \sqrt{w_f} \qquad \text{.............(18)}$$

where f_1 = coefficient to accommodate changes in wind velocity pressure (See Table 14).
f_2 = coefficient to accommodate changes in pressure coefficients (See Table 15).
f_3 = coefficient to accommodate the pier spacing, if any (See Table 16).
f_4 = coefficient to accommodate pier configuration, if any (See Table 17).

[The formulae given in Figure 7, upon which Table 17 is based (n =12), assume that pier sections are solid. The filling of cavities in piers formed of hollow units with mortar or grout increases the section modulus in the case of an elastic analysis, and the size of the stress block in a plastic analysis.]

The panels which are supported by piers may be designed in accordance with 7.3.

Table 14: Coefficient for free stream velocity pressure (f_1)

Zone	Terrain Category			
	1	2	3	4
1	0,66	0,74	1,00	1,03
2	0,62	0,70	0,94	0,96
3	0,58	0,64	0,87	0,89
4	0,52	0,58	0,78	0,81

Table 15: Coefficient for pressure coefficient (f_2)#

Wall type	Location of wall section from a discontinuity	Wall without a return at discontinuity	Wall with a return at discontinuity
Long wall (L* ≥ 4h)	0 to 2h	0,72§	0,81
	2h to 4h	0,84	0,93
	>4h	1,00	1,00
Short wall (L* < 4h)	—	0,84	0,84

§ Wall is assumed to terminate in a pier
# L* = Overall length of wall between discontinuities

Table 16: Coefficient for pier spacing (f_3)#

Height of wall above ground, m	Pier spacing, m*													
	1,2	1,5	1,8	2,1	2,4	2,7	3,0	3,6	4,2	4,8	5,4	6,0	6,5	7,0
1,5	0,96	0,88	0,82	0,78	0,75	0,74	0,73	0,73	0,73	0,73	np	np	np	np
1,8	0,95	0,86	0,80	0,75	0,72	0,70	0,68	0,67	0,67	0,67	0,67	0,67	np	np
2,1	0,94	0,85	0,78	0,74	0,70	0,67	0,65	0,62	0,62	0,62	0,62	0,62	0,62	0,62
2,4	0,93	0,84	0,77	0,73	0,69	0,66	0,63	0,60	0,58	0,57	0,57	0,57	0,57	0,57
2,7	0,93	0,84	0,77	0,72	0,68	0,65	0,62	0,58	0,56	0,54	0,54	0,54	0,54	0,54
3,0	0,93	0,83	0,76	0,71	0,67	0,64	0,61	0,57	0,54	0,53	0,51	0,51	0,51	0,51

np = not permitted
* Pier spacing $\nless$ (12 + m)t
\# f_3 = 1,0 if wall has no piers

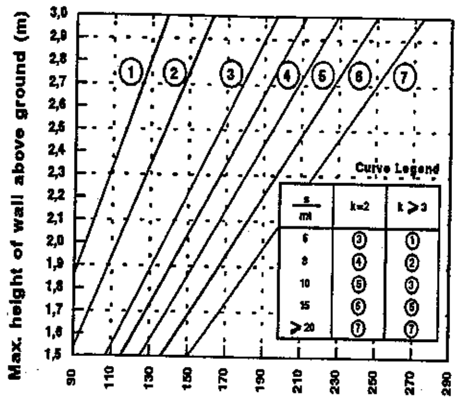
Table 17: Coefficient for pier type (f_4) #

Pier type/ wall type	k	Elastic analysis (uncracked section)				Plastic analysis (cracked analysis)			
		m				m			
		1	2	3	4	1	2	3	4
No piers	1	1,00				1,00			
Piers projecting on one side only (minimum value) *	2	1,00	1,00	1,10	1,19	1,00	1,17	1,24	1,29
	3	1,07	1,37	1,58	1,73	1,24	1,43	1,58	1,70
	4	1,39	1,82	2,11	2,32	1,44	1,74	1,97	2,14
	5	1,74	2,28	2,64	2,90	1,66	2,07	2,37	2,61
Piers projecting on both sides	2	1,00	1,00	1,10	1,17	1,18	1,35	1,43	1,48
	3	1,00	1,25	1,44	1,58	1,61	1,81	1,92	2,01
	4	1,21	1,58	1,84	2,05	1,98	2,23	2,40	2,52
	5	1,45	1,93	2,27	2,53	2,32	2,64	2,86	3,03
Z shaped wall	2	1,47	1,51	—	—	1,43	1,47	—	—
	3	2,17	2,24	—	—	1,82	1,92	—	—
	4	2,77	2,88	—	—	2,18	2,35	—	—
	5	3,31	3,46	—	—	2,52	2,76	—	—
Diaphragm	3	2,95	2,95	—	—	2,45	2,49	—	—
	4	3,76	3,78	—	—	2,89	2,98	—	—
	5	4,47	4,51	—	—	3,30	3,44	—	—

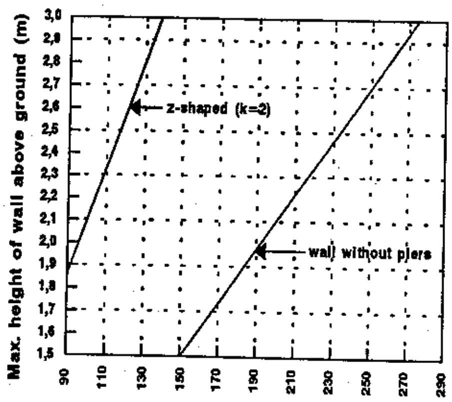
* See calculation sheet 5 on page 35 for values of maximum value
\# Pier spacing $\nless$ (12 + m)t

6.3 Limiting dimensions

The height of walls without piers, walls with piers projecting on one or both sides, and of Z-shaped walls with k =2 is restricted by the limiting dimensions calculated from Equation (14) (see 5.8). These limits are presented graphically in Figure 8.



Walls with piers projecting on one or both sides



Walls without piers and z-shaped walls

NOTE : **No height limitations are placed on diaphragm walls less than 3,0m in height, and on z-shaped walls with k=3 or greater.**

Figure 8 : Maximum height of wall based on effective wall thickness

The horizontal spacing of piers is governed by the limiting dimensions given in 5.8, and by the height to length ratio of the wall panel. These conditions may be satisfied by selecting a pier spacing within the limits given in Table 16, and by ensuring that any vertical control joints provided are located in accordance with Figure 9.

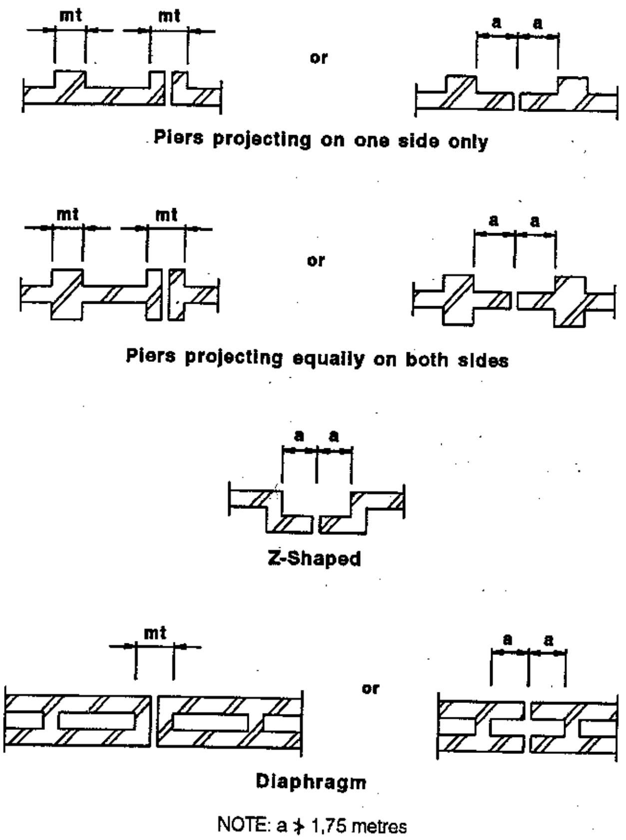


Figure 9 : Position of control joints

6.4 Shear

Equation (8) may be expressed as: $v^* = \dfrac{VQ}{t\,w_f}$ ....(19)

where Q is a shear stress distribution coefficient proportional to n, m and k at a particular point in the cross section of a solid wall. Values of Q (n = 12) at flange/rib interfaces and at the point where the shear stress is a maximum are tabulated in Table 18.

It can be shown that, in the case of walls without piers of solid or hollow unit construction, the maximum shear stress is always within acceptable limits, provided that the height of the wall is within the prescribed limits given in Figure 7. Diaphragm and Z-shaped walls, as well as walls with piers, should satisfy the following relationship:

$$t \geq \frac{3{,}29\ v_1\ v_2\ v_3\ v_4\ v_5}{w_f} \qquad \text{............(20)}$$

where t is in millimetres and;
v_1 = coefficient to accommodate changes in wind pressure (see Table 19).
v_2 = coefficient to accommodate changes in pressure coefficients (see Table 20).
v_3 = coefficient to accommodate pier spacing (see Table 21).
v_4 = coefficient to accommodate pier configuration (see Table 22).
v_5 = coefficient to accommodate type of bonding (see Table 23).
w_f = effective flange width, in metres

In the case of diaphragm walls, where flat metal ties are used to tie the webs to the flanges, the following relationship should be satisfied in addition to equation (20):

$$n^* \geq \frac{41{,}5\ v_1\ v_2\ v_3\ v_4\ m}{r\,t_s\,w_f} \qquad \text{............(21)}$$

where n* is the number of ties provided per metre of rib to resist shear across the rib-flange interface, and r and t_s is expressed in millimetres and w_f in metres.

Table 18: Shear stress distribution coefficients (Q)

Pier type (see Figure 7)	k	Q for shear stress at flange/web				Q for maximum shear stress			
		m				m			
		1	2	3	4	1	2	3	4
No piers	1	1,50				1,50			
Piers projecting on one side only	2 3 4 5	5,76 5,98 4,84 3,89	3,97 3,44 2,62 2,04	3,13 2,49 1,84 1,41	2,63 2,00 1,45 1,10	5,76 5,98 4,84 3,89	3,97 3,44 2,62 2,07	3,13 2,49 1,86 1,46	2,63 2,00 1,48 1,16
Piers projecting on both sides	2 3 4 5	2,93 4,00 3,85 3,42	2,25 2,55 2,25 1,92	1,88 1,94 1,65 1,40	1,64 1,60 1,34 1,13	2,93 4,00 3,85 3,42	2,25 2,55 2,25 1,92	1,88 1,94 1,65 1,40	1,64 1,60 1,34 1,13
Z shaped wall	2 3 4 5	9,75 5,97 4,10 3,06	5,25 3,20 2,17 1,60	— — — —	— — — —	9,75 6,07 4,29 3,31	5,25 3,30 2,35 1,83	— — — —	— — — —
Diaphragm	3 4 5	5,98 4,13 3,12	3,21 2,21 1,65	— — —	— — —	6,04 4,24 3,25	3,27 2,31 1,78	— — —	— — —

Table 19: Coefficient for free stream velocity pressure (v_1)

Zone	Terrain category			
	1	2	3	4
1	2,27	1,81	1,00	0,94
2	2,58	2,06	1,14	1,08
3	3,00	2,41	1,33	1,25
4	3,67	2,94	1,63	1,53

Table 20: Coefficient for pressure coefficient (v_2)#

Wall type	Location of wall section from discontinuity (see Figure 5)	Wall without a return at discontinuity	Wall with a return at discontinuity
Long wall ($L^* \geq 4h$)	0 to 2h 2h to 4h >4h	1,92 1,42 1,00	1,54 1,17 1,00
Short wall ($L^* < 4h$)	—	1,42	1,42

L^* = Overall length of wall between discontinuities

Table 21: Coefficient for pier spacing (v_3)#

Height of wall above ground, m	Pier spacing, m													
	1,2	1,5	1,8	2,1	2,4	2,7	3,0	3,6	4,2	4,8	5,4	6,0	6,5	7,0
1,5	1,44	1,69	1,89	2,05	2,16	2,23	2,25	2,25	2,25	2,25	np	np	np	np
1,8	1,80	2,14	2,43	2,68	2,88	3,04	3,15	3,24	3,24	3,24	3,24	3,24	np	np
2,1	2,16	2,59	2,97	3,31	3,60	3,85	4,05	4,32	4,41	4,41	4,41	4,41	4,41	4,41
2,4	2,52	3,04	3,51	3,94	4,32	4,66	4,95	5,40	5,67	5,76	5,76	5,76	5,76	5,76
2,7	2,88	3,49	4,05	4,57	5,04	5,47	5,85	6,48	6,93	7,20	7,29	7,29	7,29	7,29
3,0	3,24	3,94	4,59	5,20	5,76	6,28	6,75	7,56	8,19	8,64	8,91	9,00	9,00	9,00

np = not permitted
# v_3 = h if wall has no piers, where h is in metres

Table 22: Coefficient for pier configuration (v_4)

Pier type (see Figure 7)	k	v_4 m=1	2	3	4
No piers	1	1,50			
Piers projecting on one side only	2 3 4 5	5,76 5,98 4,84 3,89	3,97 3,44 2,62 2,07	3,13 2,49 1,86 1,46	2,63 2,00 1,45 1,16
Piers projecting on both sides	2 3 4 5	2,93 4,00 3,85 3,42	2,25 2,55 2,25 1,92	1,88 1,94 1,65 1,40	1,64 1,60 1,34 1,13
Z shaped wall	2 3 4 5	9,75 6,07 4,29 3,31	5,25 3,30 2,35 1,83	— — — —	— — — —
Diaphragm	3 4 5	6,04 4,24 3,25	3,27 2,31 1,78	— — —	— — —

Table 23: Coefficient for type of pier/rib bonding (v_5)

Pier type (see Figure 7)	v_5
Diaphragm — ribs tied with flat metal ties — fully bonded ribs	 1,0 1,0
Z-shaped wall (fully bonded)	1,0
Piers projecting equally on both sides — fully bonded piers — piers tied with wire ties	 1,0 1,4
Piers projecting on one side only — fully bonded piers — piers tied with wire ties	 1,0 1,4

6.5 Control joints

10 to 12 mm wide control joints should be provided at intervals of between 5 and 7 m. Control joints should generally be located either midway between piers or ribs, or through the centre of the rib or pier, as shown in Figure 9. In the case of Z-shaped walls, control joints may be placed only midway between the ribs so as to ensure that the assumptions upon which the section properties are based remain valid.

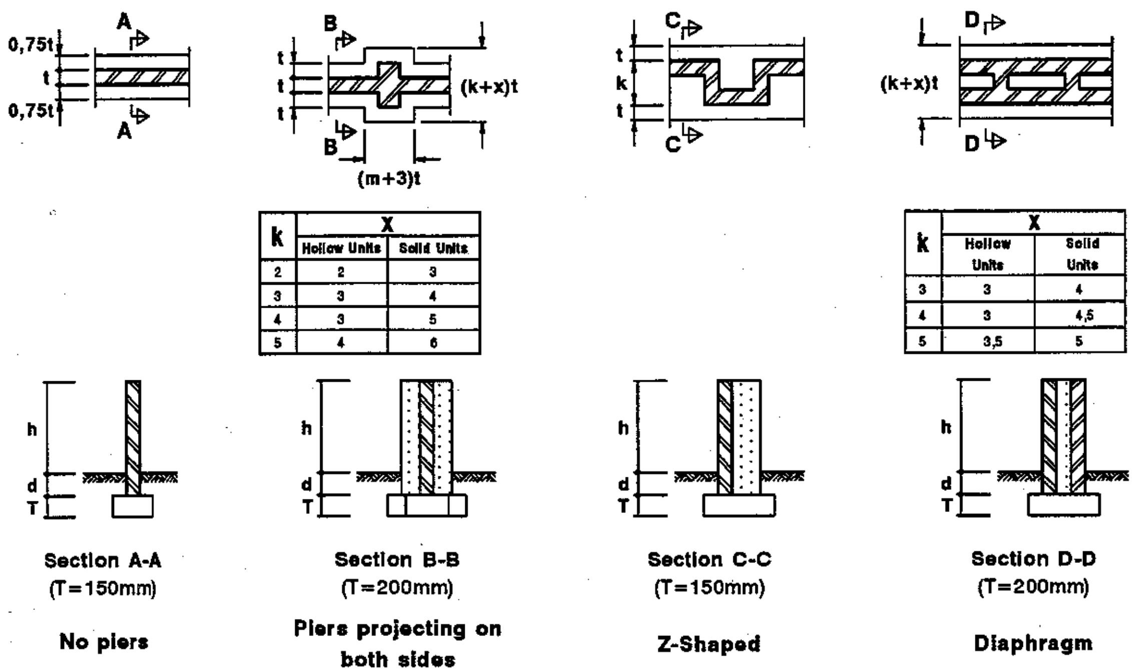
No reduction in flange width need be contemplated (see 5.3) where joints are provided midway between the piers and ribs, since the foundation will provide adequate restraint at the sections where the maximum values of bending moments occur.

6.6 Foundations

The maximum permissible bearing pressure, based on an elastic analysis, can be calculated from the following equation:

$$f = \frac{N}{A_S} \pm \frac{M}{Z_f} \qquad \text{.............(22)}$$

If the applied moment, axial force and the foundation dimensions are expressed as a function of the wall or flange thickness, t, it can be shown that f is independent of t. The dimensions of foundations supporting walls which either have no piers or have symmetrical pier configurations, where the maximum bearing pressure does not exceed 50 kPa, are given in Figure 10. Walls which have piers projecting on one side only are subject to eccentricities due to the wall geometry. Such walls should be designed in accordance with recognised engineering practice.



k	X	
	Hollow Units	Solid Units
2	2	3
3	3	4
4	3	5
5	4	6

k	X	
	Hollow Units	Solid Units
3	3	4
4	3	4,5
5	3,5	5

Note: • **Piers projecting on one side to be designed in accordance with recognised engineering practice**
• **Minimum bearing pressure = 50kPa**

Figure 10 : Foundation details

7. AN APPROACH TO THE ROUTINE DESIGN OF REINFORCED FREE-STANDING WALLS

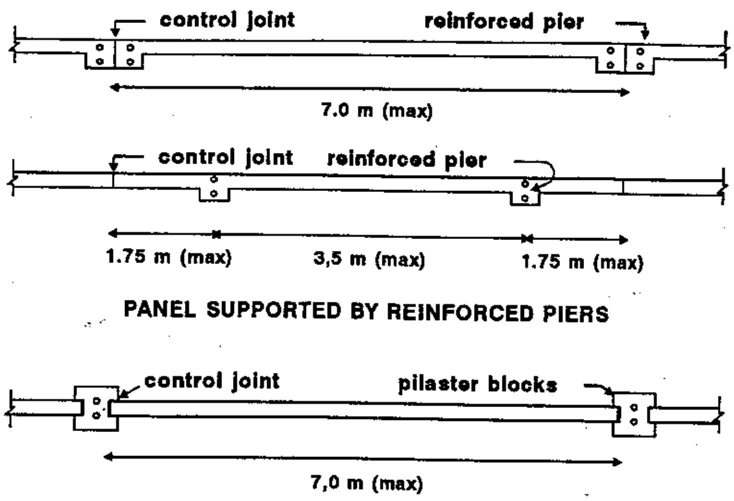
7.1 Basic assumptions

In the derivation of the design aids and charts which follow, the following assumptions are made:

- No damp-proof courses are present in walls
- Class II mortar is used (see Section 2.3)
- Sands either comply with SABS 1090 or may be described as *good* or *average* in terms of the PCI test (see Section 2.2.3)
- The minimum nominal compressive strengths of concrete masonry solid and hollow units are 10,5 MPa and 7,0 MPa, respectively
- Concrete masonry units, in addition to complying with SABS 1215, have the SABS mark (see Section 5.5.2)
- The minimum densities of concrete masonry units are as listed in Table 8
- Walls do not retain soil or other materials exerting horizontal loading
- Walls are not located in places of public assembly
- Walls either terminate in piers or are fully bonded to returns
- High yield reinforcing bars are used
- Infill concrete is Grade 30 or better
- Piers are spaced at equal intervals not exceeding 7,0 m

7.2 General arrangements

Different configurations of reinforced free-standing walls comprising an unreinforced wall panel and reinforced piers/plasters are illustrated in Figure 11. Depending on the pier spacing, control joints may be formed at either the ends of panels or at the centres of alternate wall panels at intervals not exceeding 7,0 m (refer to 4.2). As a variation to the wall panels illustrated in Figure 11, wall panels may be staggered to form Z-shaped walls, the rib between the panels being reinforced so as to constitute a reinforced pier.



PANEL SUPPORTED BY REINFORCED PILASTERS

Figure 11 : Basic panel configurations

The condition of support restraint exerted by piers/pilasters on wall panels may be described as being either pinned or fixed. Panels terminating in pilaster blocks may be regarded as being pinned. Panels containing piers within their length between control joints, and panels terminating in fully bonded piers, may be regarded as being fully fixed at their supports.

The size of a wall panel is governed by its lateral strength and the dimensions of the panel itself. On the other hand, the height of a free-standing reinforced wall is generally governed by foundation considerations, particularly stability against overturning. Reinforced strip footings designed to resist overturning tend to be unduly large due to the fact that reinforced free-standing walls are considerably more slender and hence lighter than structurally equivalent unreinforced walls. In many instances, the provision of reinforced concrete piers (mini-piles) beneath the reinforced masonry pilasters/piers designed as laterally loaded, partly buried poles to transmit the lateral loads to the ground, may well prove to be more economical.

7.3 Panel design

Design calculations for panels can be facilitated by equating the expression for M_A and M_E given in Equations (4) and (5), and solving for W_k which would then represent the maximum wind loading which may be applied to a specific wall panel for given values of H and L, and for a given masonry unit. It can be shown, on the basis of yield line theory, that the maximum wind load (W_R) which a wall panel is capable of resisting is given by:

$$W_R = \frac{Bt^2}{\gamma_f} \qquad \text{.............(23)}$$

Where B is a coefficient which is dependent on the type of masonry unit and support conditions for a given panel. (see Tables 24 to 26).

Table 24: Value of B for panel supported on two adjacent sides +

Height of wall above foundation	Length of wall panel for different unit types, m #							
	Solid		90 mm hollow		140 mm hollow		190 mm hollow	
m	1,5	1,75	1,5	1,75	1,5	1,75	1,5	1,75
1,8	159	126	111	88	97	78	83	67
2,1	150	117	104	82	91	72	78	62
2,4	145	112	99	77	88	68	75	58
2,7	141	108	96	74	85	65	72	56
3,0	138	105	94	72	83	63	71	54
3,3	136	103	92	70	81	62	69	53

+ panel supported by reinforced piers
# length of wall panel = distance between centre line of pier / pilaster and vertical free edge at control joint.

Table 25: Value of B for panel supported on three sides — pinned ends+

Unit type	Height of wall above foundation m	Length of wall panel, m #								
		3,0	3,5	4,0	4,5	5,0	5,5	6,0	6,5	7,0
Solid units	1,8	96	78	67	58	52	47	43	40	37
	2,1	85	71	60	52	45	41	37	35	32
	2,4	78	64	55	47	41	37	34	31	28
	2,7	74	60	50	44	38	34	30	28	26
	3,0	70	56	47	40	36	32	28	26	24
	3,3	67	54	44	38	33	30	27	24	22
90 mm Hollow units	1,8	69	57	49	43	38	35	32	30	28
	2,1	61	51	43	37	33	30	28	25	24
	2,4	56	47	39	34	30	27	24	22	21
	2,7	52	42	36	31	27	24	22	20	19
	3,0	49	40	33	29	25	23	20	19	17
	3,3	47	38	31	27	24	21	19	17	16
140 mm Hollow units	1,8	60	50	43	37	33	30	28	26	24
	2,1	54	45	38	33	29	26	24	22	21
	2,4	49	41	34	30	26	23	21	20	18
	2,7	46	37	32	27	24	21	19	18	16
	3,0	43	35	29	25	22	20	18	16	15
	3,3	41	33	27	24	21	18	17	15	14
190 mm Hollow units	1,8	52	43	37	32	29	26	24	22	21
	2,1	46	38	32	28	25	23	21	19	18
	2,4	42	35	29	25	22	20	18	17	16
	2,7	39	32	27	23	21	18	17	15	14
	3,0	37	30	25	22	19	17	15	14	13
	3,3	35	28	24	20	18	16	14	13	12

+ panels supported by reinforced pilasters
# length of wall panel = pier / pilaster spacing

Table 26: Value of B for panel supported on three sides — fixed ends +

Unit type	Height of wall above foundation m	Length of wall panel, m #								
		3,0	3,5	4,0	4,5	5,0	5,5	6,0	6,5	7,0
Solid units	1,8	150	121	102	89	77	69	62	57	53
	2,1	139	111	93	79	70	62	56	51	46
	2,4	131	104	86	73	63	57	51	46	42
	2,7	125	99	81	68	59	52	47	42	39
	3,0	121	94	77	64	55	49	44	39	36
	3,3	117	91	74	62	53	46	41	37	33
90 mm Hollow units	1,8	106	87	74	64	56	50	45	42	39
	2,1	98	79	66	57	50	45	40	37	34
	2,4	92	73	61	52	46	41	36	33	30
	2,7	87	69	57	48	42	38	34	30	28
	3,0	84	66	54	45	39	35	31	28	26
	3,3	81	63	51	43	37	33	29	27	24
140 mm Hollow units	1,8	93	76	65	56	49	44	40	37	34
	2,1	86	69	58	50	44	39	35	32	30
	2,4	81	64	53	45	40	36	32	29	27
	2,7	77	60	50	42	37	33	29	27	24
	3,0	74	58	47	40	34	30	27	25	23
	3,3	71	56	45	38	33	29	25	23	21
190 mm Hollow units	1,8	80	65	56	48	42	38	34	31	29
	2,1	73	59	50	43	38	34	30	28	26
	2,4	69	55	46	39	35	31	27	25	23
	2,7	65	52	43	36	31	28	25	23	21
	3,0	63	49	40	34	29	26	24	21	19
	3,3	61	47	39	32	28	24	22	20	18

+ panels supported by reinforced piers
# length of wall panel = pier / pilaster spacing

Thus, provided that the applied wind load W_K does not exceed W_R, the strength of the wall panel will be adequate.

However, $$W_K = F_1 \; W_Q \qquad \text{.............(24)}$$

where F_1 is the wind pressure coefficient given in Table 27 and W_Q is the free stream velocity pressure given in Table 10.

Equating W_R and W_K in Equations (23) and (24)

$$t \geq \sqrt{\frac{\Upsilon_f F_1 W_Q}{B}} \qquad \text{.............(25)}$$

It can be demonstrated that, for the full range of panels given in Tables 24 to 26, the maximum shear stress determined in accordance with Equation (5) satisfies Equation (4).

Table 27: Wind pressure coefficient (F_1)#

Wall type	Location of wall section from a discontinuity	Wall without a return at discontinuity	Wall with a return at discontinuity
Long wall ($L^* \geq 4h$)	0 to 2h 2h to 4h >4h	2,3 1,7 1,2	1,85 1,4 1,2
Short wall ($L^* < 4h$)	—	1,7	1,7

L^* = Overall length of wall between discontinuities; h = height of wall above ground

7.4 Lateral loading on piers/pilasters

The load shed from the wall panels on to the piers/pilasters may be assessed on the assumption that the panel load disperses at 45° with respect to the supports. The lateral bending moment exerted on a pier/pilaster is therefore dependent on the panel geometry — as the H:L ratio of the panel reduces, the moment increases, and attains a maximum constant value when $L \geq 2H$

The maximum bending moment on pilasters/piers due to the lateral loading of wall panels can be expressed as:

$$M_{RP} = \Upsilon_f \; W_K \,(F_{1\,\text{LEFT PANEL}} + F_{1\,\text{RIGHT PANEL}})\; C \qquad \text{.............(26)}$$

where C is a coefficient which is dependent on the panel geometry (see Table 28). (F_1 = O if no panel exists on the side of the pier/pilaster under consideration).

Table 28: Pier/pilaster moment coefficient (C) for panels

Height of wall above foundation m	Pier / pilaster spacing								
	3,0	3,5	4,0	4,5	5,0	5,5	6,0	6,5	7,0
1,8	1,87	1,94	1,94	1,94	1,94	1,94	1,94	np	np
2,1	2,75	2,97	3,08	3,09	3,09	3,09	3,09	3,09	3,09
2,4	3,76	4,15	4,43	4,58	4,61	4,61	4,61	4,61	4,61
2,7	4,91	5,49	5,96	6,30	6,51	6,56	6,56	6,56	6,56
3,0	6,19	6,98	7,67	8,23	8,65	8,91	9,00	9,00	9,00
3,3	7,61	8,64	9,56	10,35	11,01	11,51	11,84	11,97	11,98

np - not permitted

Likewise, the maximum shear force on pilaster/piers due to the lateral loading of wall panels can be expressed as:

$$V_{RP} = \Upsilon_f \; W_K \,(F_{1\,\text{LEFT PANEL}} + F_{1\,\text{RIGHT PANEL}})\; D \qquad \text{.............(27)}$$

where D is a coefficient which is dependent on the panel geometry (see Table 29). (F_1 = O if no panel exists on the side of the pier/panel under consideration.)

Table 29: Pier/pilaster shear force coefficient (D) for panels

Height of wall above foundation m	Pier / pilaster spacing								
	3,0	3,5	4,0	4,5	5,0	5,5	6,0	6,5	7,0
1,8	1,58	1,62	1,62	1,62	1,62	1,62	1,62	1,62	1,62
2,1	2,03	2,14	2,20	2,21	2,21	2,21	2,21	2,21	2.21
2,4	2,48	2,67	2,80	2,88	2,88	2,88	2,88	2,88	2,88
2,7	2,93	3,19	3,40	3,54	3,63	3,65	3,65	3,65	3,65
3,0	3,38	3,72	4,00	4,22	4,38	4,47	4,50	4,50	4,50
3,3	3,83	4,24	4,60	4,89	5,13	5,29	5,40	5,44	5,45

7.5 Design of reinforced piers/pilasters

In order to reduce the number of variables in the design of reinforced piers/pilasters, the following assumptions may be made:

- All infill comprises a Grade 30 concrete with a minimum cement content of 300 kg/m³
- Reinforcement comprises uncoated carbon steel bars or wires with a minimum yield strength of 450 MPa and a diameter not exceeding 20 mm
- The minimum concrete cover to reinforcement in infills is 40 mm except in the case of reinforced 140 mm hollow units where the cover is reduced to 30 mm. (In practice, this means that piers constructed with 140 mm hollow units may not be used in industrial environments and coastal areas up to 1 km from the coast line, whereas the only restriction placed on piers constructed with other types of units, and on pilasters, is that they may not be used in tidal and splash zones, unless protected from corrosion.[31])

In Figure 12, a typical range of reinforced piers/pilasters, as well as design information such as the effective depth and section width of piers/pilasters is presented. All the physical dimensions required for design purposes may be obtained from Figure 12. Reinforced piers/pilasters may then be designed by simply ensuring that the values of M_{RP} and V_{RP} calculated in accordance with equation (26) and (27), respectively, is less than the value of M_d and V_d given in Table 30, for a selected pier configuration.

Table 30: The moment and shear resistance reinforced piers and pilasters (M_d, V_d)

Pier configuration § (Refer to Figure 12)	Resistance moment (M_d), kNm								Shear resistance (V_d), kN							
	Reinforcement in each face															
	1Y10	1Y12	1Y16	1Y20	2Y10	2Y12	2Y16	2Y20	1Y10	1Y12	1Y16	1Y20	2Y10	2Y12	2Y16	2Y20
90 mm solid units																
290 x 390	6,4	8,4	9,3	9,3	np	np	np	np	13,4	13,7	14,4	15,4	np	np	np	np
390 x 390	6,7	9,1	12,5	12,5	11,6	12,5	12,5	12,5	17,7	18,1	18,8	19,8	18,4	19,0	20,6	22,6
290 x 490	9,5	12,9	18,2	18,2	np	np	np	np	18,4	18,8	19,5	20,5	np	np	np	np
390 x 490	9,8	13,5	21,3	24,5	17,7	23,1	24,5	24,5	24,6	24,9	25,6	26,6	25,3	25,9	27,4	29,4
106 mm solid units																
330 x 440	7,5	10,0	13,3	13,3	np	np	np	np	16,9	17,2	17,9	18,9	np	np	np	np
440 x 440	7,8	10,7	16,5	17,7	13,9	17,7	17,7	17,7	22,2	22,5	23,3	24,3	22,9	23,5	25,1	27,1
330 x 550	10,9	14,9	23,4	25,7	np	np	np	np	23,2	23,5	24,3	25,3	np	np	np	np
440 x 550	11,2	15,5	25,2	34,3	20,6	27,6	34,3	34,3	30,7	31,0	31,8	32,8	31,4	32,0	33,5	35,5
140 mm solid units																
440 x 590	11,7	16,4	27,2	38,8	22,0	30,0	46,0	46,6	31,5	31,8	32,6	33,5	32,2	32,8	34,3	36,3
590 x 590	11,7	16,7	28,3	41,4	22,7	31,4	50,3	62,5	42,0	42,3	43,1	44,0	42,7	43,3	44,8	46,8
140 mm hollow units																
140 x 290	5,2	6,2	6,2	6,2	np	np	np	np	6,1	6,4	7,1	8,1	np	np	np	np
290 x 290	6,0	8,3	12,6	12,8	10,7	12,8	12,8	12,8	11,9	12,2	12,9	13,9	12,5	13,1	14,7	16,7
140 x 540	13,0	17,7	27,2	28,3	np	np	np	np	12,2	12,5	13,3	14,3	np	np	np	np
290 x 540	13,7	19,3	32,3	46,4	26,0	35,7	55,2	58,5	24,5	24,8	25,6	26,6	25,2	25,8	27,4	29,4
190 mm hollow units																
190 x 390	8,2	11,0	14,4	14,4	np	np	np	np	10,8	11,1	11,9	12,9	np	np	np	np
390 x 390	8,8	12,3	20,1	28,0	16,4	22,0	29,5	29,5	21,5	21,8	22,6	23,6	22,2	22,8	24,3	26,3
190 x 590	14,3	19,8	31,7	39,4	np	np	np	np	17,5	17,8	18,6	19,5	np	np	np	np
390 x 590	14,7	21,1	35,8	52,5	28,7	39,8	63,8	64,0	35,2	35,5	36,2	37,2	35,8	36,4	38,0	40,0
pilasters																
260 x 260 (internal)	6,1	8,8	15,6	23,7	12,3	17,5	29,5	43,0	10,2	10,5	11,3	12,3	10,9	11,5	13,1	15,1

§ - pier configuration expressed as pier width x pier depth
np - not permitted

REINFORCED PIERS - SOLID UNITS

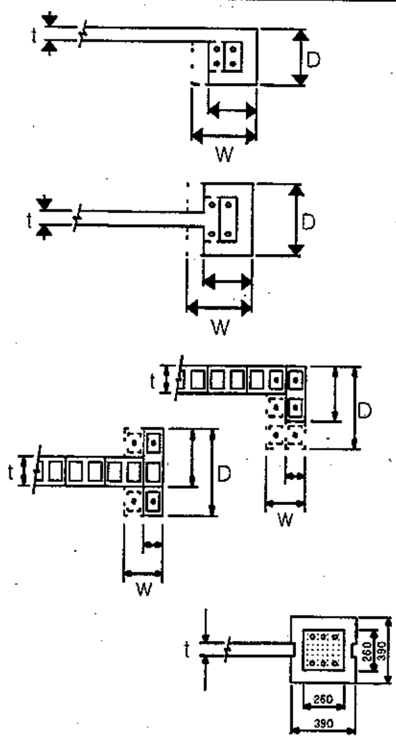
UNIT WIDTH (t)	PIER SIZE (WxD)	EFFECTIVE DEPTH (d)	No. of BARS
90	290x390	250	2
	390x390	250	4
	290x490	350	2
	390x490	350	4
106	330x440	280	2
	440x440	280	4
	330x550	390	2
	440x550	390	4
140	440x590	400	2 or 4
	590x590	400	4

REINFORCED PIERS - HOLLOW UNITS

UNIT WIDTH (t)	PIER SIZE (WxD)	EFFECTIVE DEPTH (d)	No. of BARS
140	140x290	220	2
	290x290	220	4
	140x540	470	2
	290x540	470	4
190	190x390	305	2
	390x390	305	4
	190x590	505	2
	390x590	505	4

PILASTER

PIER SIZE	EFFECTIVE DEPTH (d)	No. of BARS
390x390	210	2 to 6

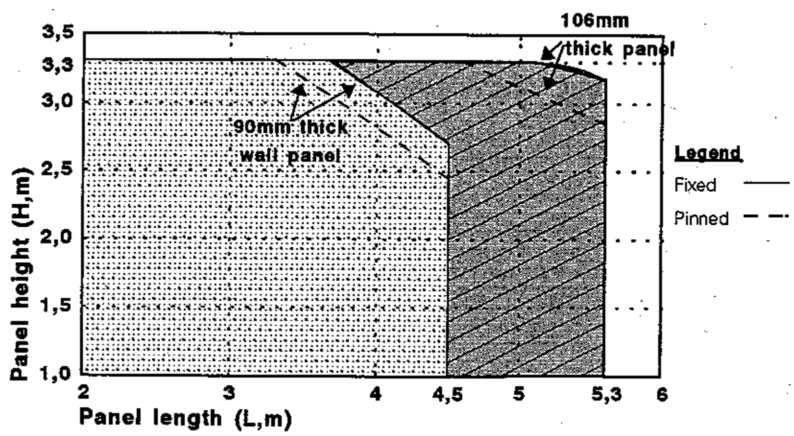


NOTE: *Cover to steel = 40mm (min) or in the case of 140mm hollow units = 30mm

*All piers, with the exception of those constructed using 140mm hollow units, may be used in all above ground exterior environments, with the exception of tidal splash zones.

*Piers constructed using 140mm hollow units may not be used in industrial zones and within 1km from the coast line.

FIGURE 12 - Typical Pier / Pilaster Details



NOTE: For walls of thickness 140mm and greater, H ≯ 3,3 and L ≯ 7,0

Figure 13 : Maximum size of wall panel supported on three sides

7.6 **Limiting dimensions**

Provided that the area of panels with three simply supported (pinned) edges and of continuous panels with fixed edges does not exceed $1350t^2$ and $1500t^2$, respectively, and no horizontal dimension exceeds 50 t the requirements governing limiting dimensions will be satisfied. Similarly, provided that the length of panels supported along two sides does not exceed 1,75 m, the horizontal limiting dimension of 20 t stipulated in Section 5.8 will not be exceeded.

Panels whose overall dimensions are within the limits of Tables 24 to 26 and, in the case of panels with a panel thickness of 90 or 106mm, within the shaded portion of Figure 13, will satisfy the abovementioned criteria.

7.7 **Design of foundations**

No simplified procedure can be readily devised for the design of foundations for reinforced free-standing walls, as is the case for unreinforced free-standing walls. Consequently, such foundations should be designed in accordance with established engineering practice.

8. SIMPLIFIED PROCEDURES FOR THE DESIGN OF UNREINFORCED AND REINFORCED FREE-STANDING WALLS

8.1 Unreinforced walls

The maximum height of unreinforced free-standing walls of a given thickness and pier configuration may be readily determined using the aforementioned design charts in the following manner:

Step 1: Determine the wind velocity pressure zone from Figure 3 and select the terrain category from Table 9.

Step 2: Verify that the distance between the selected terrain and an upwind dissimilar terrain is adequate, so as to ensure that the choice of terrain category is in fact valid, and that local topographical features will not cause an increase in wind loading (refer to 5.6.3).

Step 3: Obtain the coefficient for free stream velocity pressures (f_1) and pressure coefficients (f_2) from Tables 14 and 15 respectively .

Step 4: Calculate the effective flange width of piers (w_f) in metres where $w_f = (12 + m)t$.
Note: $w_f = 1,0$ if wall has no piers.

Step 5:	Determine the value of t_{eq} from the appropriate curve in Figure 6 using the desired value of h as the abscissa.	Estimate h and derive the coefficient for pier spacings (f_3) and the coefficient for pier configuration (f_4) from Tables 16 and 17.
Step 6:	Calculate the product of $f_3\ f_4\ \sqrt{w_f}$ for a given wall or flange thickness: $\sqrt{w_f}\ f_3\ f_4 = \dfrac{t_{eq}}{f_1\ f_2\ t}$	Calculate the equivalent wall thickness t_{eq} using the following equation for a given wall or flange thickness, t: $t_{eq} = f_1\ f_2\ f_3\ f_4\ t\ \sqrt{w_f}$
Step 7:	Select the pier spacing and pier configuration, and verify that the product of $f_3\ f_4\ \sqrt{w_f}$ given in Tables 16 and 17 is greater than the value calculated in Step 5.	Determine the value of h from the appropriate curve in Figure 6, using t_{eq} as calculated in step 6 as the ordinate and confirm the value of f_3.

Step 8: Should the depth of soil cover above the foundation exceed 0,3m, reduce h by the excess amount.

Step 9: Ensure that the wall panel between piers is adequate. (Refer to steps 5 and 6 in 8.2).

Step 10: Ensure that h is within the limits given in Figure 8.

Step 11: Determine the shear coefficients for the free-stream velocity pressure (v_1), the pressure coefficient (v_2), the pier spacing (v_3), the pier configuration (v_4) and the type of rib/flange bonding (v_5) from Tables 19 to 23 respectively.

Step 12: Ensure that in all wall types:
$$t \geq \frac{3{,}29\ v_1\ v_2\ v_3\ v_4\ v_5}{w_f}$$
and, in the case of diaphragm walls with metal ties between the flange and rib that:
$$n^* \geq \frac{41{,}5\ h\ v_1\ v_2\ v_3\ v_4\ m}{r\ t_s\ w_f}$$

where w_f is expressed in metres, t, r and t_s in milimetres and n* in number of wall ties provided per linear metre.

Step 13: Ensure that control joints are provided at intervals not exceeding 5 to 7 metres at the positions shown in Figure 9.

Step 14: Determine the dimensions of foundations for walls without piers or walls having symmetrical pier configurations in accordance with Figure 10 with recognised engineering practice.

8.2 Reinforced walls

Free-standing walls with reinforced masonry piers or pilasters may be designed using the following procedure:

Step 1: Determine the wind velocity pressure zone from Figure 3 and select the terrain category from Table 9.

Step 2: Verify that the distance between the selected terrain and an upwind dissimilar terrain is adequate, so as to ensure that the choice of terrain category is in fact valid, and that local topographical features will not cause an increase in wind loading (refer to 5.6.3).

Step 3: Obtain the value of W_Q and F_1 from Tables 10 and 27, respectively.

Step 4: Select the panel and pier configuration (see Figure 11), and the coefficient B for the appropriate support condition from Tables 24, 25 or 26.

Step 5: Ensure that, in the case of panels having a panel thickness of 90 or 106 mm, the selected dimensions are within the shaded portion in Figure 13.

Step 6: Ensure that, in the case of panels: $t \geq 1\,000 \sqrt{\dfrac{1{,}2\ F_1 W_Q}{B}}$ (t in millimetres)

Step 7: Obtain the coefficient C and D for each panel supported by a pier/pilaster from Tables 28 and Table 29, respectively.

Step 8: Calculate the applied moment (M_{RP}) and shear (V_{RP}) acting on the piers/pilasters:

$$M_{RP} = 1{,}2 \quad W_K \ (F_{1\ \text{LEFT PANEL}} + F_{1\ \text{RIGHT PANEL}})\ C$$

$$V_{RP} = 1{,}2 \quad W_K \ (F_{1\ \text{LEFT PANEL}} + F_{1\ \text{RIGHT PANEL}})\ D$$

Step 9: Select the pier configuration from Figure 12 and ensure that M_{RP} and V_{RP} does not exceed M_d and V_d as derived from Tables 30 and 31.

Step 10: Design the foundation and its reinforcement in accordance with recognised engineering practice.

9. GENERAL REQUIREMENTS APPLICABLE TO FREE-STANDING WALLS DESIGNED IN ACCORDANCE WITH THE SIMPLIFIED PROCEDURES

A check-list of general requirements relating to the construction of free-standing walls follows. The general and specific notes prescribed below should be embodied in the construction drawings or in the specifications pertaining to the walls in question.

9.1 General notes.

9.1.1 Concrete masonry units to comply with SABS 1215 and have the SABS mark.

9.1.2 Solid and hollow units to have nominal compressive strengths of 10,5 and 7,0 MPa, respectively.

9.1.3 Sand to comply with SABS 1090.

9.1.4 Cement to be OPC, PC 15 or Masonry Cement.

9.1.5 Lime to comply with Class A2P of SABS 523.

9.1.6 Mortar to be Class II in accordance with SABS 0164 Part 1.

9.1.7 Horizontal and perpend joints to be solidly filled with mortar. Bed joint thickness not to exceed 13mm.

9.1.8 *No earth to be retained by the wall.*

9.1.9 *No horizontal damp proof courses (dpc) to be provided.*

9.1.10 *Piers of adequate dimensions to be provided to support all gates.*

9.1.11 *Height of masonry erected in a day not to exceed 1,5 m.*

9.2 Specific notes pertaining to unreinforced walls.

9.2.1 *Collar-joints in multi-leaf walls to be solidly filled with mortar. 3,15 mm diameter PWD or crimp wire ties to be provided at vertical and horizontal intervals not exceeding 450 mm. Alternatively, headers to be provided at vertical intervals not exceeding 600 mm.*

9.2.2 *Piers to be tied into walls with headers at least every fourth course. Alternatively, 3,15 mm diameter PWD or crimp wire ties to be provided for every 200 mm width at vertical intervals not exceeding 300 mm. All collar joints to be solidly filled with mortar.*

9.2.3 *All cavities in hollow masonry units forming piers to be solidly filled with mortar, grout or infill concrete as the work proceeds.*

9.2.4 *10 mm wide control joints to be provided at intervals not exceeding 7 m and at large steps in foundations. Position of joints to be in accordance with Figure 9.*

9.2.5 *Piers to extend to top of wall without any reduction in their cross-sectional dimensions.*

9.2.6 *Foundations to be in accordance with Figure 10. (Permissible bearing pressure: 50 kPa.)*

9.2.7 *Concrete mix to foundations to consist of cement:sand:19 mm stone in the proportion by volume of 1:4:4.*

9.2.8 *Walls to terminate either in a return wall or in a pier.*

9.3 Specific notes pertaining to reinforced walls

9.3.1 *Specified cover to reinforcement to be maintained by means of approved spacers.*

9.3.2 *Reinforcement to be free from mud, oil, grease, paint, loose rust and mill scale.*

9.3.3 *All hollow units to be laid on a full bed of mortar.*

9.3.4 *All excess mortar and mortar droppings to be removed from cores/cavities prior to filling.*

9.3.5 *Infill concrete to consist of cement : sand : 13 mm stone in the proportion by volume of 1: 2,5: 2,0.*

9.3.6 *Infill concrete to be placed, in layers as the masonry work proceeds, in vertical lifts not exceeding 450 mm or in the case of hollow units, 900 mm , to within 50 mm of the level of the last course laid, and to be compacted by means of a rod immediately after placing.*

9.3.7 *Reinforcement to consist of high yield steel bars complying with SABS 920 and bent in accordance with SABS 82.*

9.3.8 *Bars or wire not exceeding 6 mm in diameter to be placed in alternate bed joints in piers for the purpose of fixing vertical reinforcement in position.*

9.3.9 *10mm wide control joints are to be provided in accordance with Figure II.*

10. WORKED EXAMPLES OF MASONRY WALL DESIGN

Worked examples illustrating the use of the simplified procedures for both unreinforced and reinforced free-standing walls follow hereafter. It should be noted that the design of foundations for unreinforced walls with unsymmetrical pier configurations and reinforced walls is beyond the scope of this design guide.

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CALCULATIONS - (Project/Subject:)	Proj.No.	CMA	By	RBW
WORKED EXAMPLES ON THE STRUCTURAL DESIGN OF MASONRY WALLS.	Date	10/9/93	Chkd	BAL
	Sheet	1 of 8	Appd	RBW

QUESTION: What is the maximum height permitted for the following walls located in a Category 3 wind terrain on the Natal South Coast?

Natal South Coast is situated in Zone 2

$f_1 = 0{,}94$

$f_2 = 1{,}0$ max; $0{,}81$ min, (assuming wall with a return at its end).

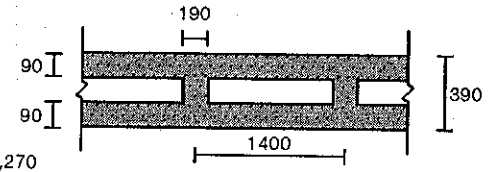
Figure 3
Tables 9,14
Table 15

1) 90 mm wide hollow unit diaphragm wall with an overall wall thickness of 390 mm (Rib = 190 mm at 1,4 m centres).

$k = \frac{390}{90} = 4{,}3$

$m = \frac{190}{90} = 2{,}1$

$w_f = \frac{190 + 12 \times 90}{1000} = 1{,}270$



Assume $h_{max} = 2{,}7$; then $f_3 = 0{,}87$

$f_4 = 3{,}99$ (uncracked section).

$t_{eq} = 0{,}94 \times f_2 \times 0{,}87 \times 3{,}99 \times 90 \times \sqrt{1{,}270} = f_2 \times 331$

$t_{eq\ (max)} = 1{,}0 \times 331 = 331$; **thus h_{max} = 2,7m (away from corner)**

$t_{eq\ (min)} = 0{,}81 \times 331 = 268$; **thus h_{max} = 2,0m (at corner with return)**

Table 16
Table 17
Series (2) curves Figure 6

CHECKS:

* Limitations on wall height : No limitations
* Shear- If 20 x 3 mm flat metal ties provided:

$t > \frac{3{,}29 \times 1{,}14 \times 1{,}54 \times 2{,}17 \times 2{,}31 \times 1{,}0}{1{,}270} = 23$ mm c.f. 90

Number of ties/m $> \frac{41{,}5 \times 1{,}14 \times 1{,}54 \times 2{,}17 \times 2{,}31 \times 2{,}1}{20 \times 3 \times 1{,}27} = 10{,}1$

therefore provide two ties at every course. (10 ties/m)

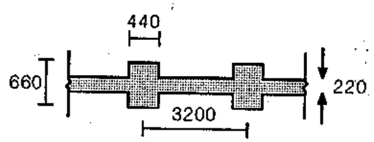
* Panel check by inspection - acceptable

Figure 8
Tables 19 to 23
Equation 20
Equation 21

2) 106 mm wide solid unit, 220 mm wide wall with 660 mm x 440 mm piers projecting on both sides. (Piers at 3,2 m centres).

Thus $k = \frac{660}{220} = 3$; and $m = \frac{440}{220} = 2$

$w_f = \frac{440 + 12 \times 220}{1000} = 3{,}08$



Assume $h_{max} = 3{,}0$; then $f_3 = 0{,}60$

f_4 (uncracked) = 1,25

f_4 (cracked) = 1,81

Thus $t_{eq} = 0{,}94 \times f_2 \times 0{,}60 \times f_4 \times 220 \times \sqrt{3{,}08} = 218\ f_2 f_4$

Table 16
Table 17
Equation 18

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CALCULATIONS - (Project/Subject:)	Proj.No.	CMA	By	RBW
WORKED EXAMPLES (CONTINUED)	Date	10/9/93	Chkd	BAL
	Sheet	2 of 8	Appd	RBW

	Cracked	Uncracked	Remarks
$t_{eq\ max}$	395 mm	281 mm	If h < 3,0; f_3 increases
$t_{eq\ min}$	320 mm	235 mm	If h < 3,0; f_3 increases
h_{max}	**3,0 m**	**2,5 m**	**Panel away from corners**
h_{min}	**3,0 m**	**2,0 m**	**Panel adjacent to corner**

Series (1) curves Figure 6

CHECKS:

* Limitations on wall height:

$\frac{s}{mt} = \frac{3200}{2 \times 220} = 7{,}3$; Thus no restriction.

* Shear - If piers are tied with wire ties to flange:

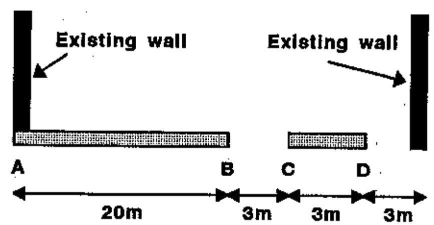
$t > \frac{3{,}29 \times 1{,}14 \times 1{,}54 \times 7{,}02 \times 2{,}55 \times 1{,}4}{3{,}08} = 47$mm cf 220mm provided

* Panel check: $B = 107$; $t \geq 1000 \times \sqrt{\frac{1{,}2 \times 1{,}85 \times 0{,}365}{107}} = 87$ cf 220mm provided

Figure 8 curve (1)
Tables 19 to 23
Tables 10, 26 and 27

DESIGN EXAMPLE

Design a 2,5 m high (minimum) wall for the following site situated in a well developed suburb in Johannesburg.



1) **Unreinforced option**

$f_1 = 1{,}0$ (Category 3, Region 1)

Location	f_2
Wall AB	
0 to 5 m from A	0,81
5 m to 10 m from A	0,93
> 10 m from A	1,0
0 to 5 m from B	0,72
5 to 10 m from B	0,84
> 10 m from A	1,0
Wall C - D	0,84

Table 15

Assume solid units with clinker aggregates. Using Figure 6 where h = 2,5m :

Figure 6 (Series 1 curves)

$t_{eq\ (uncracked)\ required} = 278$ mm

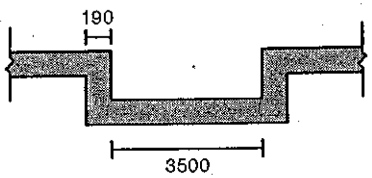
$t_{eq\ (cracked)\ required} = 283$ mm

Try 190 mm thick (90 mm units) Z-shaped wall with piers at 3,500 m centres

$m = \frac{190}{190} = 1,0$

$k = \frac{590}{190} = 3,1$

$W_f = \frac{190 + 12 \times 190}{1000} = 2,47$



$h_{max} = 2,5$; therefore $f_3 = 0,60$ — Table 16

$f_{4\ (elastic)} = 2,23$; $f_{4\ (plastic)} = 1,86$ — Table 17

Thus using minimum value of f_2

$t_{eq\ (uncracked)} = 1,0 \times 0,72 \times 0,60 \times 2,23 \times 190 \times \sqrt{2,47} = 288$ cf 278

$t_{eq\ (cracked)} = 1,0 \times 0,72 \times 0,60 \times 1,86 \times 190 \times \sqrt{2,47} = 240$ cf 283

Thus wall is adequate on the basis of an elastic analysis.

Calculation of permissible max height of wall C-D and central portion of wall A-B ≥ 5,0 m away from B.

Uncracked section: $t_{eq} = 1,0 \times 0,84 \times 0,57 \times 2,23 \times 190 \times \sqrt{2,47} = 319$ — Figure 6 (Series 1 curve)

Thus h_{max} = 3,0m

CHECKS:

* Limitation on height: k = 3; therefore no limitations w.r.t. height. — Figure 8
* Shear: Piers are fully bonded.

Consider maximum value for v_2 — Tables 19 to 23

$t \geq \frac{3,29 \times 1,0 \times 1,92 \times 5,67 \times 5,89 \times 1,0}{2,47} = 85$ mm cf. 190 mm provided. — Equation 20

* Panel check: B = 91 (H = 3,3; L = 3,5)

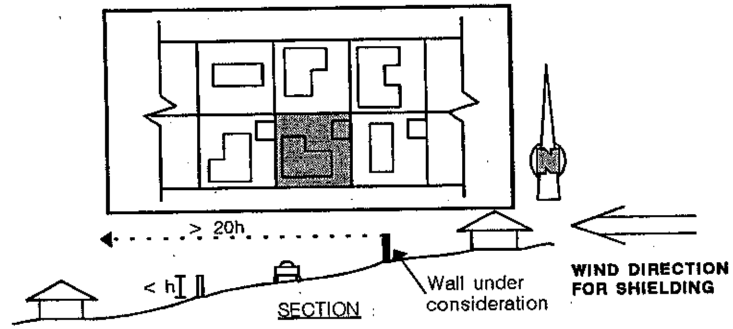
$t_{min} = 1000 \times \sqrt{\frac{1,2 \times 2,3 \times 0,32}{91}} = 99$mm .cf.190mm provided — Tables 10, 26 and 27

Effects of shielding

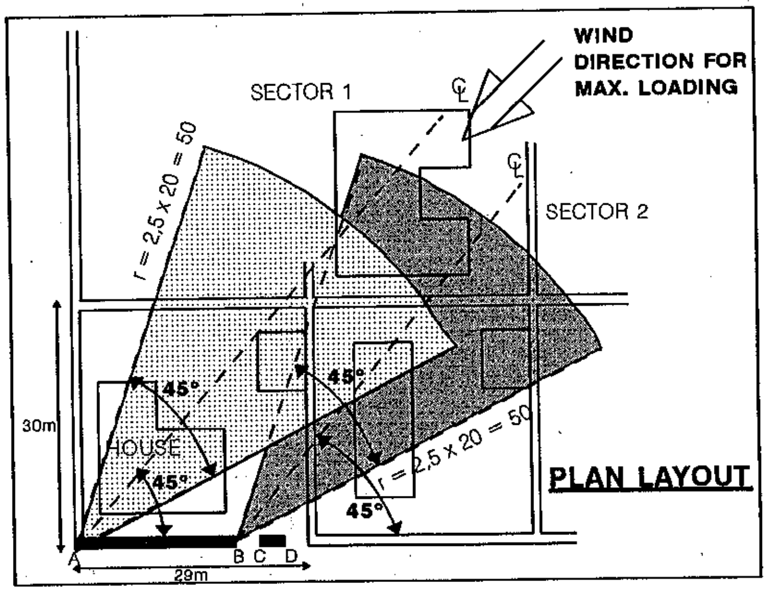
Shielding from surrounding structures could significantly influence the geometry of the wall.

The maximum wind coefficient occurs at 45 degrees w.r.t. the wall. Consequently 45 degree sectors of radius 20h centered along this line should be considered.

Example: assume wall under consideration is located in a neighbourhood as shown below. — Appendix A



Note: Shielding only applies if the heights of structures on the windward side of the wall within a 45° sector of radius 20h are greater than the height of the wall. In the above example, shielding only occurs on the northern side of the wall.



Shielding effects for wind blowing from a south westerly direction.

sector 1 $n = 4$; $b_s = 9$ $h_s = 3,0$

sector 2 $n = 4$; $b_s = 8$ $h_s = 3,0$

Thus $D_{sector1} = \dfrac{2,5\ (^{10}/_4 + 5)}{\sqrt{3,0 \times 9}} = 3,6$

Thus $D_{sector2} = \dfrac{2,5\ (^{10}/_4 + 5)}{\sqrt{3,0 \times 8}} = 3,8$

Therefore $f_{s1} = 1,21$ and $f_{s2} = 0,68$ — Appendix A Table A

In this example, if symmetrical wall configurations are employed, shielding effects will not bring about an economy in design. However, if an unsymmetrical configuration is adopted ie: piers projecting on one side only, economies can be effected.
Table 17 tabulates the minimum values of f_4 for piers projecting on one side only. The maximum values, however, can be derived from Figure 7 and are tabulated below for convenience.

Maximum value of f_4 for piers projecting on one side only.

k	m							
	uncracked section				cracked section			
	1	2	3	4	1	2	3	4
2	1,30	1,45	1,55	1,62	1,52	1,64	1,68	1,71
3	1,93	2,21	2,36	2,45	2,10	2,23	2,30	2,35
4	2,63	2,96	3,12	3,23	2,59	2,75	2,85	2,94
5	3,29	3,64	3,83	3,95	3,02	3,23	3,37	3,49

(f_4 min)

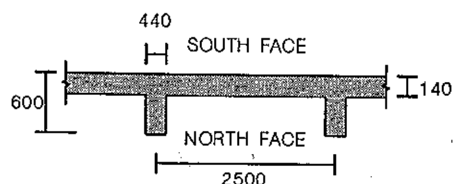
(f_4 max)

Thus consider a 140mm solid unit, 140mm wide wall with 600 x 440mm piers projecting on the north side only. (Piers at 2,5m centres).

$m = \dfrac{440}{140} = 3,1$

$k = \dfrac{600}{140} = 4,3$

$w_f = \dfrac{440 + 12 \times 140}{1000} = 2,12$



$h_{max} = 2,5$: therefore $f_3 = 0,68$ — Table 16

Thus for uncracked section: $f_{4\ max} = 3,34$; $f_{4\ min} = 2,29$
cracked section: $f_{4\ max} = 3,02$; $f_{4\ min} = 2,07$ — Table 17 and previous table

Thus using minimum f_2 and f_4

$t_{eq\ uncracked} = 1,0 \times 0,68 \times 0,72 \times 2,29 \times 140 \times 1,21 \times \sqrt{2,15} = 278$ cf. 278

$t_{eq\ cracked} = 1,0 \times 0,72 \times 0,68 \times 2,07 \times 140 \times 1,21 \times \sqrt{2,15} = 252$ cf. 283

Thus wall is adequate on the basis of an elastic analysis.

Note: if a constant height of 2,5m is desired, the pier spacing for the central portion of the wall 5m away from the corners can be increased to 3,0m

$f_3 = 0,63$

t_{eq} uncracked $= 1,0 \times 0,84 \times 0,63 \times 2,29 \times 140 \times 1,21 \times \sqrt{2,15} = 301$.cf.278 — Table 16

<u>CHECKS:</u>

* Limitation on height:

$k = 3;\quad \dfrac{s}{mt} = \dfrac{3,0}{3 \times 0,14} = 7,1$ — Figure 8 curve (2)

$h_{max} = 2,9$ c.f. 2,5

* Shear - If piers are tied with wire ties to flange:

$t > \dfrac{3,29 \times 1,0 \times 1,92 \times 4,68 \times 1,72 \times 1,4 \times 0,68}{2,15} = 23$ mm c.f.140mm provided — Tables 19 to 23

* Panel check: B = 129 (H = 2,8; L = 2,5)

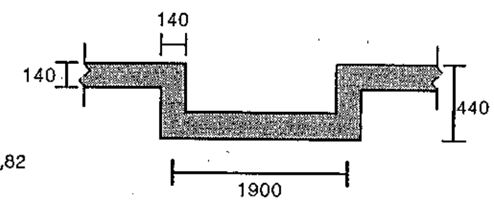
$t_{min} = 1000 \times \sqrt{\dfrac{1,2 \times 2,3 \times 0,32}{124}} = 84$.cf.140mm provided — Tables 10, 26 and 27

Note: If the wall was shielded about both faces, a 140mm wide Z-shaped wall with piers at 1,90 centres would have sufficed.

$m = \dfrac{140}{140} = 1,0$

$k = \dfrac{440}{140} = 3,1$

$W_f = \dfrac{140 + 12 \times 140}{1000} = 1,82$



$h_{max} = 2,5$ therefore $f_3 = 0,76$ — Table 16

$f_4 = 2,23$ (uncracked section) — Table 17

$t_{eq\ uncracked} = 1,0 \times 0,72 \times 0,76 \times 2,23 \times 140 \times 1,21 \times \sqrt{1,82} = 279$ cf 278

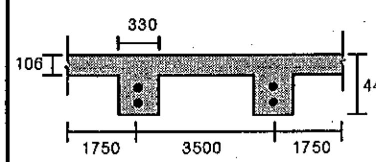
2) **Reinforced option**

W_q = 0,32 kN/m² (Category 3, Region 1) — Table 10

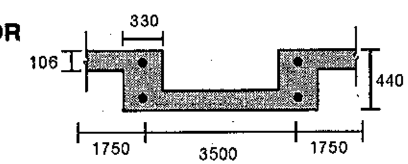
Location	F_1
Wall AB	
0 to 5m from A	1,85
5m to 10m from A	1,4
> 10m from A	1,2
0 to 5m from B	2,3
5 to 10m from B	1,7
> 10m from A	1,2
Wall C-D	1,7

Table 27

Assume wall is constructed using 106mm wide solid units and 330x440mm reinforced piers (Control joints provided at 7,0m centres)



OR



Wall height above foundation = 2500 + 300 = 2800

Using the maximum value of F_1

*** Wall panel checks**

Panel supported on two sides: (B = 107; H = 2,8; L = 1,75) — Table 24

thus $t_{min} = 1000 \times \sqrt{\dfrac{1,2 \times 2,3 \times 0,32}{107}}$ = 91mm cf 106mm provided

Panel supported on three sides: B = 97; (H = 2,8; L = 3,5) — Table 26

thus $t_{min} = 1000 \times \sqrt{\dfrac{1,2 \times 2,3 \times 0,32}{97}}$ = 95mm cf 106mm provided

Note: 106mm wide panel within the limits of Figure 13

*** Pier checks**

C = 5,99 — Table 28

D = 3,37 — Table 29

Thus M_{RP} = 1,2 x 0,32 x (2,3 + 2,3) x 5,99 = 10,6 kNm

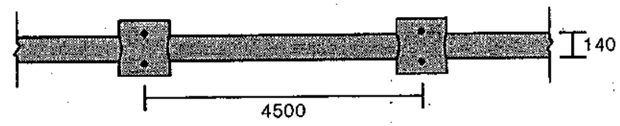
V_{RP} = 1,2 x 0,32 x (2,3 + 2,3) x 3,37 = 6,0 kN

Thus 330 x 440 pier having 1 Y16 bar in each face is adequate — Table 30

(The values of M_{RP} and V_{RP} reduce along the length of the wall as F_1 reduces and only 1 Y12 bar would be required in panels located more than 5m away from point B)

Pilaster Alternative

Consider 140mm thick wall panels constructed of solid units incorporating pilasters at 4,5m centres.



Using the maximum value of F_1

*** Wall panel checks** B = 43; (H = 2,8; L = 4,5) — Table 25

$t_{min} = 1000 \times \sqrt{\dfrac{1,2 \times 2,3 \times 0,32}{43}}$ = 143 mm cf 140mm provided

Note: t > 106mm; no check on wall panel limits is required. — Figure 13

*** Pier checks**

C = 6,94; D = 3,77 — Tables 28 and 29

Pier situated 4,5m away from B has maximum loading.

Thus M_{RP} = 1,2 x 0,32 x (2,3 + 1,7) x 6,94 = 10,7 kNm

V_{RP} = 1,2 x 0,32 x (2,3 + 1,7) x 3,77 = 5,8 kN

Thus a pilaster with 260 x 260mm internal dimensions having 2Y10 bars in each face is adequate. — Table 30

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APPENDIX A : WIND SHIELDING MULTIPLIERS

Shielding may be considered in terrain categories 3 and 4 when the height of upwind structures in a 45° sector of radius 20h, as shown in Figure A, is greater than or equal to h. The effects of shielding by such upwind structures may (according to Clause 3.2.7 and Appendix E, Clause 3.2.7, of AS 1770.2) be evaluated as follows:

Calculate the building spacing parameter (D)

$$D = \frac{h\left(\frac{10}{n} + 5\right)}{\sqrt{b_s \; h_s}}$$

where

h	=	the clear height of the wall above finished ground level.
n	=	the number of upwind shielding buildings within the sector shown in Figure A and with a height $\geq$ h.
h_s	=	the average height of shielding structures.
b_s	=	the average breadth of shielding structures normal to the windstream.

Shielding may only be considered if:

i) The heights of the upwind buildings within the sector shown in Figure A are greater than or equal to h.
ii) No "corridors" are present i.e. segments, in which no upwind shielding buildings are sited.

Unreinforced walls:

Multiply t in Step 6 of the Simplified Procedures by f_{s1} and the constants contained in the formulae provided in Step 12 by f_{s2}.

Reinforced walls:

Multiply W_k as determined in step 3 of the Simplified Procedures by the factor f_{s2} obtained from Table A.

Table A : Shielding Multiplier

D	f_{s1}	f_{s2}
$\leq$ 1,5	1,43	0,49
3,0	1,25	0,64
6,0	1,11	0,81
$\geq$ 12,0	1,00	1,00

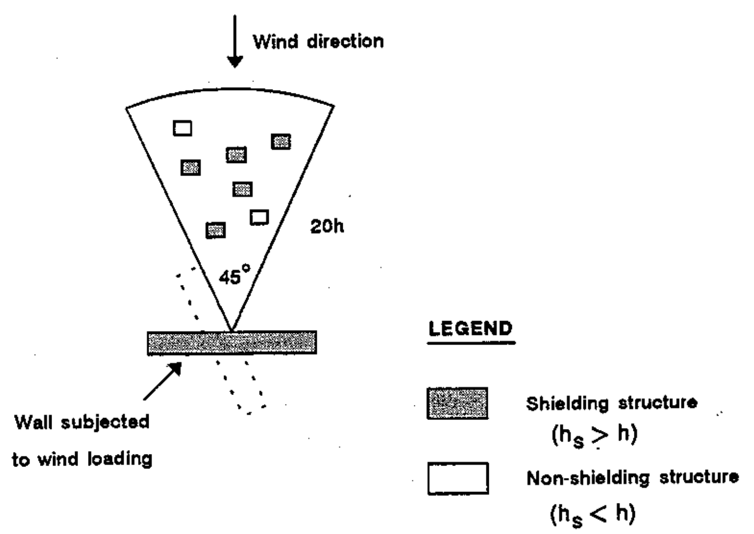

Figure A : Shielding in complex urban situations