The Structural use of masonry

SABS 0164 - Parts 1 & 2

SAICE Lecture Course

Structural Division

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THE STRUCTURAL USE OF MASONRY
SABS 0164 - PARTS 1 and 2

by

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Synopsis

Developments in the structural masonry design codes are reviewed. Critical comments on SABS 0164 - Part 1 unreinforced structural masonry are made together with reference to the new load factors arising from the revisions to SABS 0160 on loading.

General comments on the new reinforced and prestressed masonry code SABS 0164 - Part 2 are given together with worked examples of design.

1. Part 1 : Unreinforced masonry

1.1 Introduction

SABS 0164 Part 1[1] is based on the British Code of Practice, BS 5628 Part 1[2]. This code of practice was first published in 1980 and was subsequently amended during 1986 and 1987.

1.2 Amendments to Part 1

Amendments issued since 1980 have not only corrected editorial errors but have also:

i) Provided additional clauses to cover aspects such as:
   * Compliance with the National Building Regulations.
   * Movement of structural units.
   * The determination of the flexural bond strength of damp-proof courses.
   * The determination of the short term shear strength at damp-proof courses.
   * Control joints.

ii) Changed the values for characteristic flexural tensile strengths.

iii) Expanded requirements for the testing of mortars.

iv) Changed the test procedures for the determination of the flexural strength of masonry.

1.3 Proposed amendments to BS 5628 Part 1

A draft 6 amendment to BS 5628 Part 1 proposes to include the following:

i) A characteristic shear strength for brick masonry in the vertical direction of 0,7N/mm^2.
ii) Limiting the uninterrupted height of cavity walls to avoid undue loosening of wall ties due to vertical differential movements between inner and outer leaves.
iii) Values for the characteristic compressive strength of wire ties for various cavity widths.
iv) Reducing the general factor of safety to 2 in the formula provided for determining the lateral strengths of axially loaded walls and columns.
v) A design procedure for the design of propped cantilever walls.

1.4 Load factors

The SABS technical committee for Part 2 decided that the uniform load factors contained in SABS 0160[3] which in time will apply to all the other structural design codes viz concrete, steel and timber, would be used in the masonry codes. SABS 0164 - Part 1 is to be amended to take account of these changes.

The following design load combinations for the ultimate limit state should be applied:

a) Dead, wind and imposed loads

Design dead loads	= 1,5Dn
Design imposed loads	= 1,6Ln
Design wind loads	= 1,3Wn

In the particular case of free-standing walls and laterally loaded wall panels, whose removal would in no way affect the stability of the remaining structure, the design wind load may be taken as 1,2.

b) Dead, imposed and wind loads combinations

Design dead + wind load	= 0,9Dn + 1,3Wn
Design dead + imposed load	= 1,2Dn + 1,6Ln
Design dead + wind + imposed loads	
- general	= 1,2Dn + 0,5Ln + 1,3Wn
- garages, filling and storage areas	= 1,2Dn + 1,0Ln + 1,3Wn
- roofs	= 1,2Dn + 1,3Wn

1.5 The design of wall panels

In the UK, extensive research into the lateral loading of wall panels was undertaken in the 1970's[15]. The research was specifically aimed at wall panels with little or no preload, which spanned both vertically and horizontally. Prior to the publication of BS 5628 in 1978, over 100 full size walls in lengths of up to 5,5 m and heights of up to

3,6 m under various support conditions had been constructed, tested and analyzed. All of these walls were either conventional cavity walls or half-brick walls. Most of the walls that were tested were supported on three sides, with the top edge free.

The strength of masonry walls subject to lateral loading was found to depend on

- flexural strength in the orthogonal directions;
- shear strength of masonry;
- thickness of the wall;
- size and shape of the wall; and
- support configuration, i.e. the manner in which the wall spans and the degree of restraint provided by the supports.

Observations of tests on wall panels have shown that:

i) The cracking load was the maximum load that the wall resisted[15].
ii) The failure cracking pattern is reasonably similar to that which would be expected from slabs analyzed in accordance with yield line theory[17, 18].

Since the shear strength of masonry was known and understood, researchers established the flexural strength of masonry units in the two orthogonal directions. Once this had been done, the walls that had been tested were analyzed.

If a wall is assumed to behave as a plate spanning in the manner that is suggested by the support configuration and degree of restraint provided by the supports, bending moments in the walls can be obtained from one of the following analytical methods[15]:

i) Elastic plate methods (after Timoshenko)
ii) Johansen's yield line theory
iii) Finite elements

In masonry panels, the method most suitable for calculations must be able to accommodate different strengths in the two orthogonal directions and partial fixity at the supports. These two criteria are of the utmost importance if meaningful interpretations of experimental data is to be achieved. Clearly, the elastic plate method fails to meet these two criteria. The finite element method of analysis can take these two requirements into account. It is, however, a rather cumbersome technique. The only simple-to-use method, able to accommodate the abovementioned requirements, is yield line theory. A further advantage to the yield line method is that irregularly shaped walls, or walls with openings, can be readily analyzed.

Calculations [15, 10] based on elastic plate theory methods consistently underestimated the experimental failure pressure whereas those based on yield line theory give a close approximation of these values, except for panels of short length, where the failure pressures are overestimated.

Based on these experiments and analyses, moment coefficients for various three and four side configurations, calculated from yield line theory, were incorporated into Table 9 of BS 5628. The range of height to length ratios in the panels were however restricted to between 0,3 and 1,75. In an Appendix D to the code, the finite element and yield line design method, are recommended to obtain the bending moments in irregular shaped panels, or panels with openings.

More than 170 walls have been tested since 1977[16]. These tests have included walls with openings, walls supported along their top and bottom edges and along one vertical edge, walls supported on all four sides and brick walls with thicknesses of 170, 220 and 330 mm. The relationship between experimental and calculated (predicted) failure pressure, for pressures not exceeding 10 kN/m^2, is given in Figure 1.1.

The results of these latter tests, where the failure pressures obtained from yield line theory, taking into account the experimental edge restraints tend to be scattered above and below the line of prediction. This is to be expected, since mean flexural strengths, with coefficients of variation around 20% based on wallette tests, have been used in the calculations. On the other hand, the failure pressures calculated in accordance with BS 5628, using the tabulated moment coefficients (a conservative assessment) and characteristic strengths, are generally below the line of prediction. The incidence of the calculated failure pressures exceeding the experimental failure pressures is acceptable, since the calculated pressures are characteristic. By definition, this implies that up to 5% of the results may be above the line of prediction. Nevertheless, a minimum factor of safety in excess of 2,5 is maintained.

Clauses to limit both the individual panel lengths and the panel areas, were also included in the code (see Figures 1.2 and 1.3). No specific deflection criteria were defined. The handbook to the code describes the purpose of these limiting dimensions as "... essential to treat the dimensions provided by this clause checks to be carried out independently of strength design procedures. They are not design calculations in themselves but are akin to the limiting slenderness ratios for vertically loaded walls. They represent a subjective assessment based on experience of extreme panel proportions ... beyond these limits, there will be a rapidly increasing sensitivity to errors of design and construction, and sudden instability may occur[5].

The main objection by many [16] to the application of yield line theory to a brittle material, is that the assumption that moment is maintained across yield lines, is not applicable. Some writers have gone as far as to state that there is no rational basis for the application of the theory and any concurrence with experimental results should be assumed to be coincidental. Others have modified the yield line theory and have developed a fracture line theory, which takes into account the different moduli of elasticity in the two orthogonal directions. Fracture lines are assumed to develop, only when the relevant strengths are reached in the two orthogonal directions. Extremely good correlation between predicted and experimental failure pressures have been obtained in model tests [20, 21]. If the ratio of the modulus of elasticity in the two orthogonal directions is assumed to be unity, fracture line bending moment coefficients are identical to those obtained from yield line theory.

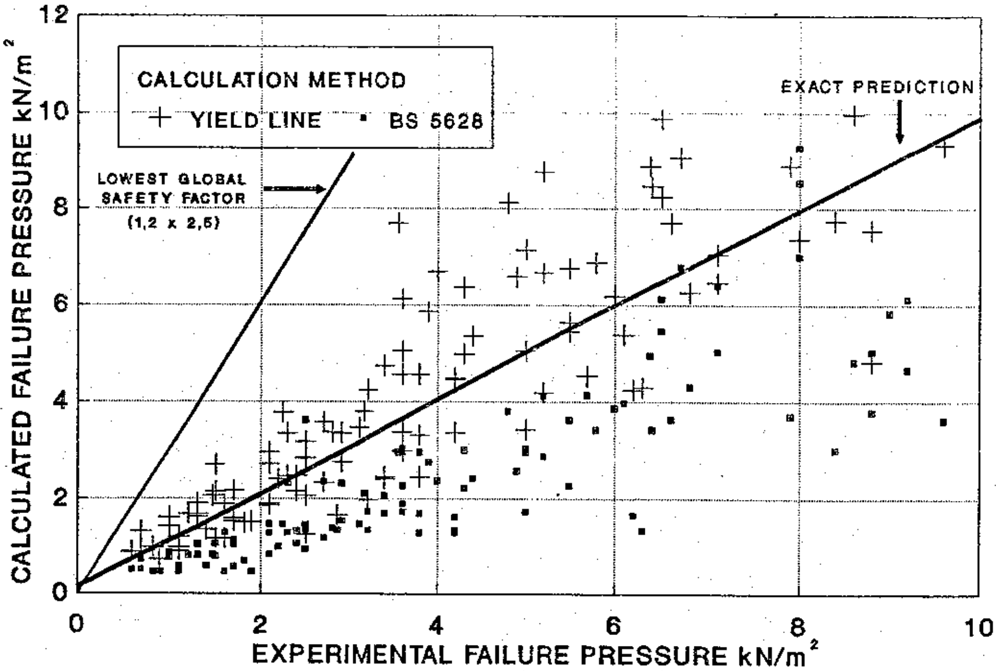
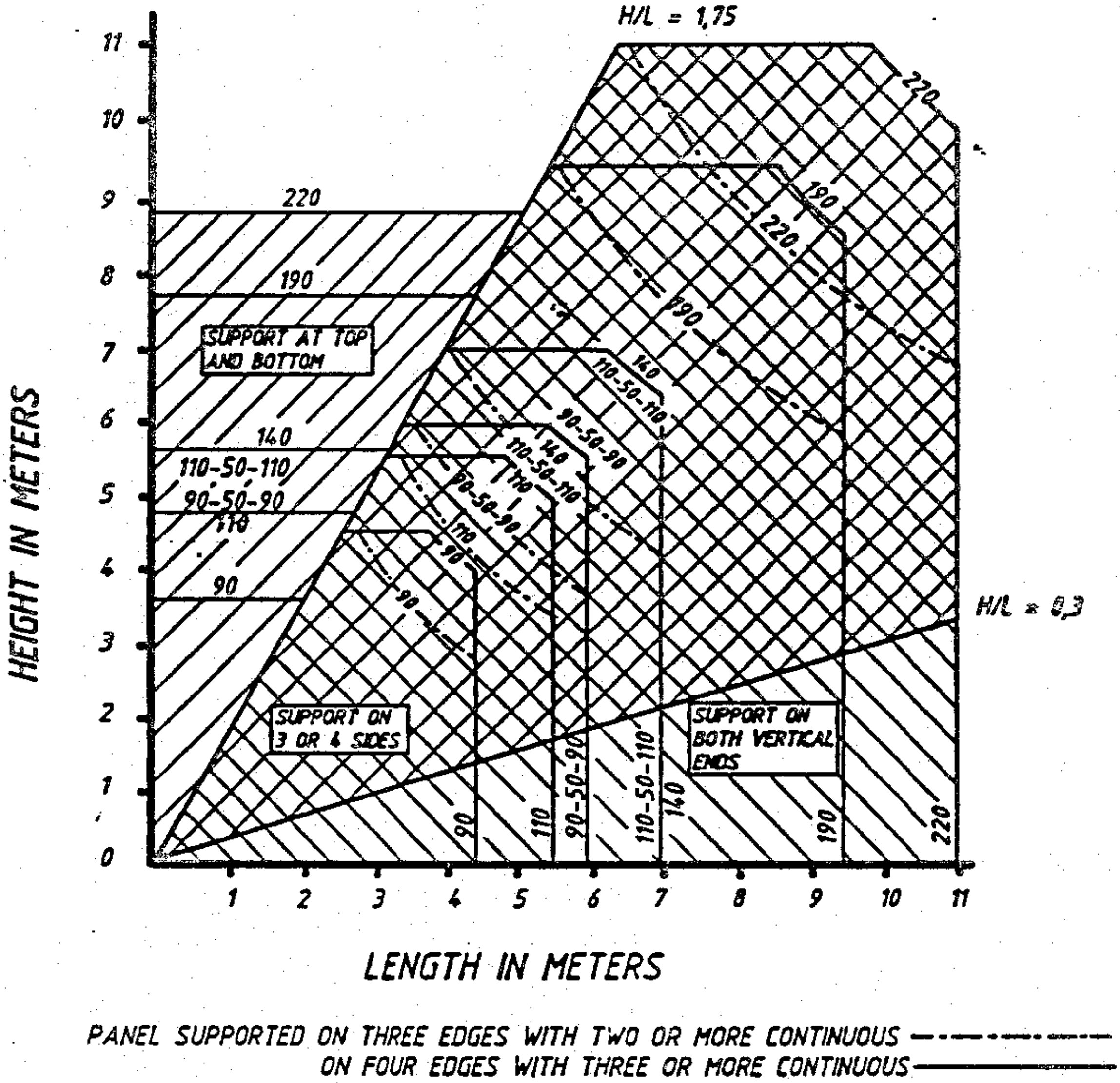
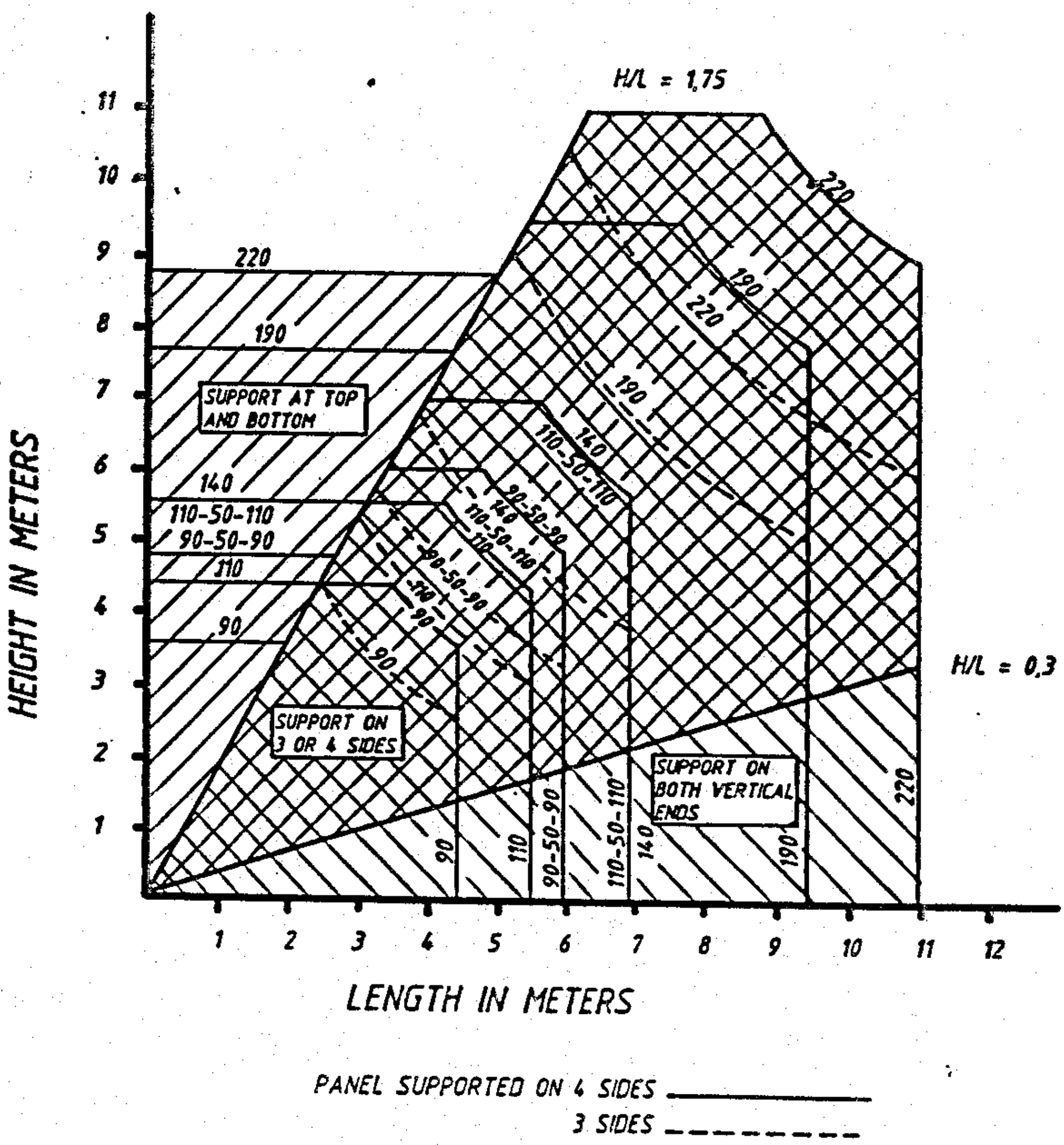


Figure 1.1: Expermental versus calculated failure pressures



NOTE: 1 t_{ef} CALCULATED IN ACCORDANCE WITH CLAUSE 5.1.2.
2 DIAGRAM IN ACCORDANCE WITH CLAUSE 5.5.2

Figure 1.2: Limiting dimensions for panels with continuity



NOTE: 1) t_{ef} CALCULATED IN ACCORDANCE WITH CLAUSE 5.1.2.3
2) DIAGRAM IN ACCORDANCE WITH CLAUSE 5.5.2

Figure 1.3: Limiting dimensions for simply supported panels

Recently, the Building Research Establishment [22], has demonstrated that, if the assumption of the production of plastic hinges in yield line theory, is replaced by considerations of the energy needed to produce a crack, the same final design strength will be achieved. In this approach, the suggested failure criterion for masonry, is the energy required to produce a unit length of crack, rather than a bending moment. Values for the energy required to produce a unit length of crack may be determined from wallette tests, whilst the theoretical energy required to produce a unit length of crack may be calculated form an energy analysis (i.e. the work equations).

Thus, in terms of this proposed theory, the existing code method for assessing the flexural design strength of a wall, based on section moduli and yield line moment coefficients, could be regarded as an analogous method to solve energy equations. The analogous equations relate the energy required to produce a unit crack length to other parameters such as wall dimensions, wall thickness and applied load in a similar manner as the soap bubble, or membrane analogy solves the complex equations for elastic torsion, or the sand heap analogy for plastic torsion. Using this analogous method yield line theory could be used to assess the strength of walls that have openings, are supported on two adjacent sides, are subjected to triangular loading etc.

Based on this theoretical approach a proposal has been made to amend the terminology and notation in BS 5628, so that the values reflected in the tables for characteristic flexural strengths and bending moment coefficients would have different descriptions and physical interpretations[22]. The major advantage of this amendment would be that:

i) calculations to determine the lateral strength of walls would be based on a more sound theory; and
ii) the code could be expanded to include design rules for walls containing openings etc.

On the other hand, the Australian code writers, being concerned with the theoretical justification of the use of yield line methods in analyzing masonry panels and the Australian experimental evidence which demonstrated that the theory significantly underestimates the strengths of panels supported on four sides, adopted an empirical strip method of analysis[23]. This method offers a more accurate assessment of panels supported on three and four sides than that of yield line theory. The strip method in the SAA Masonry Code AS 3700[10] simply summates the lateral load capacities of horizontal and vertical spanning strips[24].

AS 3700 gives bending moment coefficients for the determination the load capacities for walls in two way flexure that are supported on three (top edge free) and four sides. Formulae for assessing the vertical and horizontal moment capacities, based on flexural tensile strengths determined from either beam tests (as in the case in BS 5628), or bond wrench tests and lateral moduli of rupture (determined from beam tests on glued units[25], are provided together with requirements for robustness. This code, however, fails to give any guidance on the evaluation of walls with openings, walls with one vertical and horizontal edge free and walls of irregular shape and loading. It suggests that a one-way spanning approach may be appropriate in some cases.

1.6 Areas not adequately addressed by Part 1

1.6.1 Value of characteristic shear strength (fv)

γ_{mv}

SABS 0164 Part 1 gives a value of 2,5 and fv a value of 0,35 MPa and 0,15 MPa for class I and class II mortar respectively. However, a class II mortar is stronger than a designation (ii) mortar as defined in BS 5628 Part 1. It is therefore recommended that the value for fv given in BS 5628 Part 1, namely 0,35 MPa, be adopted for class II mortar.

A draft amendment to BS 5628 Part 1 recommends that the value of fv for brickwork in the vertical direction be increased to 0,7 MPa.

1.6.2 Evaluation of the applied shear stress

In terms of shear, the code only requires that the following relationship is satisfied:

$$V_h < \frac{fv}{\gamma_{mv}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where

$$V_h = \frac{\gamma_f V}{A} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

SABS 0164 Part 1 is based on the British code of practice BS 5628 Part 1. The handbook to BS 5628 Part 1 (5) states that equation (1) 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 compare this value against the design shear stress.

The American ACI-ASCE building code ACI 530/ASCE 5 (4) offers the following equation to determine the shear distribution across a wall:

$$v^* = \frac{\gamma_f Vay}{Ib} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

where a = area above the shear plane under consideration.
y = the distance from the neutral axis to the centroid of the area above the plan under consideration.
I = moment of inertia of the section.
b = section width at shear plane under consideration.

Formula (3), in the case of a rectangular section, may be expressed as:

$$V^* = \frac{\gamma_f 1,5V}{bt} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Formula (3) may also be used to evaluate the shear stress developed in the planes of the interface between leaves in a multi-leaf 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 joint between leaves or a leaf and a pier/rib) are provided, to ensure that wall sections have adequate vertical shear resistance so that composite action of a particular wall section takes 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 centres 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 (6) conducted laboratory measurements on the shear resistance of 36 solidly filled collar-jointed clay brick wallettes, with and without metal ties, to evaluate the shear resistance of collar-jointed walls and the influence of construction procedures and types of ties. Table 1 tabulates the results of this study.

Table 2, on the other hand, tabulates valves given for f_{kv} in other codes of practice whilst Table 3 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. AS 3700 (10), in the case of interfaces at ribs and piers, 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 1: The shear strength of collar joints

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

Table 2: Values for characteristic vertical shear strength

Codes of Practice	Value,	MPa	Remarks
AS 3700	f_{KV} 1,2p	1,25 +	Wire ties provided; $f_{KV} \ngtr 0{,}35$ MPa Masonry headers provided; fkv $\ngtr 0{,}6$ 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.

* Actual value depends on the type and condition of the interface, the consolidation of the joint and type of loading.

\# Working stress value, which incorporates factors of safety.

Table 3: Spacing of headers and ties in double leaf walls

Code of practice	Tie requirements	Header requirements
SABS 0164 Part 1 BS 5628 Part 1	Flat metal (20 x 3 mm) ties at not more than 450 horizontal and vertical centres.	One complete header course to every 5 or less stretcher courses, or the equivalent area of headers evenly distributed through the wall.
AS 3700	Cavity type ties at not more than 400 mm centres in each direction.	Header course with each alternate unit a header at 600 mm centres or less or the equivalent number of header units evenly distributed through the wall.
ACI 530/ASCE 5	2,4 to 4 ties per square meter 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 the gross sectional area is greater than 4%.

A draft amendment to BS 5628 Part 1 suggest 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 on appropriate flat metal sections, the size and spacing of which should be calculated. Roberts (7) offers the following formula to determine the shear capacity of rectangular wall ties (V_t):

$$V_t = \frac{f_y r t_s}{6\gamma m} \quad \ldots\ldots\ldots\ldots\ldots (8)$$

$$(\gamma_m = 3,0)$$

where f_y = yield stress of tie material
r = width of rectangular tie
t_s = thickness of wall tie

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

1.6.3 Wall ties

In assessing whether or not a tie is capable of transmitting the required forces between the leaves, cognizance must be taken of the load-deflection characteristics of the ties. Typical load deflection curves for some commercially available types of wall ties, clamped 75 mm apart and subjected to axial tension are shown in Figure 1.4. Apart from the 3,15 mm PWD ties, the ties in Figure 104 do not conform to SABS 28(8).

SABS 28 is only concerned with describing the material content, tie geometry and corrosion protection of wall ties. It does not address the engineering properties of the ties namely tensile and compressive strength and stiffness. By way of comparison the Australian standard, AS 2699 (9), addresses these properties, requiring that the characteristic tensile and compressive strengths be based on the lesser of the failure loads and the load producing a 1,5 mm deflection. Furthermore, it specifies ranges of acceptable wall tie stiffness for the different types of ties.

Attention must also be paid to providing a sufficient number of ties at supports to enable the reactions from the other leaf to be transmitted to the supports (see Figure 1.5).

1.6.4 Stiffness coefficients

Stiffness coefficients (K) for Z-shaped walls and diaphragm walls are not given in SABS 0164. The hand book to BS 5628 Part 1 and AS 3700 suggests that, for Z-shaped and diaphragm walls respectively, the value of K be regarded as being equal to the effective wall thickness.

1.6.5 Openings in laterally loaded wall panels

Some guidance is offered in Appendix F on the design of wall panels with openings. However, the guidance is descriptive and does not offer any formulae for specific situations.

1.6.6 Characteristic flexural strengths

Although reference is made to "bricks" and "blocks", the sizes of these units are not specified. Without these definitions, it is impossible to interpret Table 4 of Part 1. The current draft of Eurocode 6 makes no reference to bricks and blocks and simply gives values for units of different materials as shown in Table 4. It should be noted that the values for concrete bricks are lower than that given in Table 4 of Part 1 and no increase in flexural strength for high strength units is permitted.

1.6.7 DPC's

Although a test procedure is given for the determination of both the flexural and shear strengths of dpc's, no design information is offered relating to any type of dpc. To the author's knowledge, no such information is available for any of the products commonly used in South Africa.

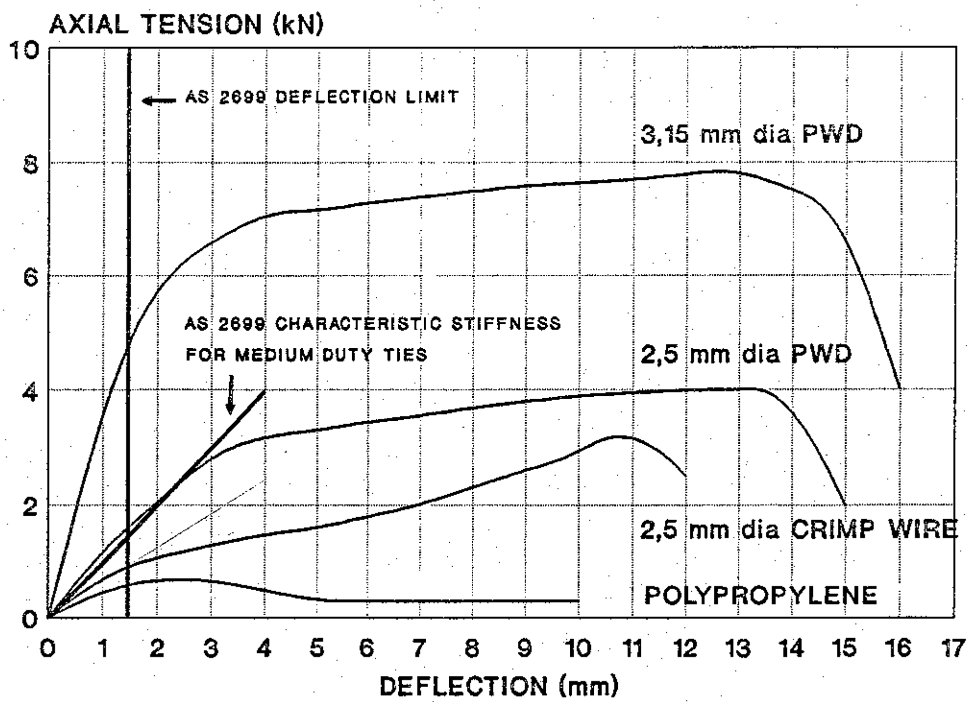


Figure 1.4: Typical load-deflection curves for cavity wall ties

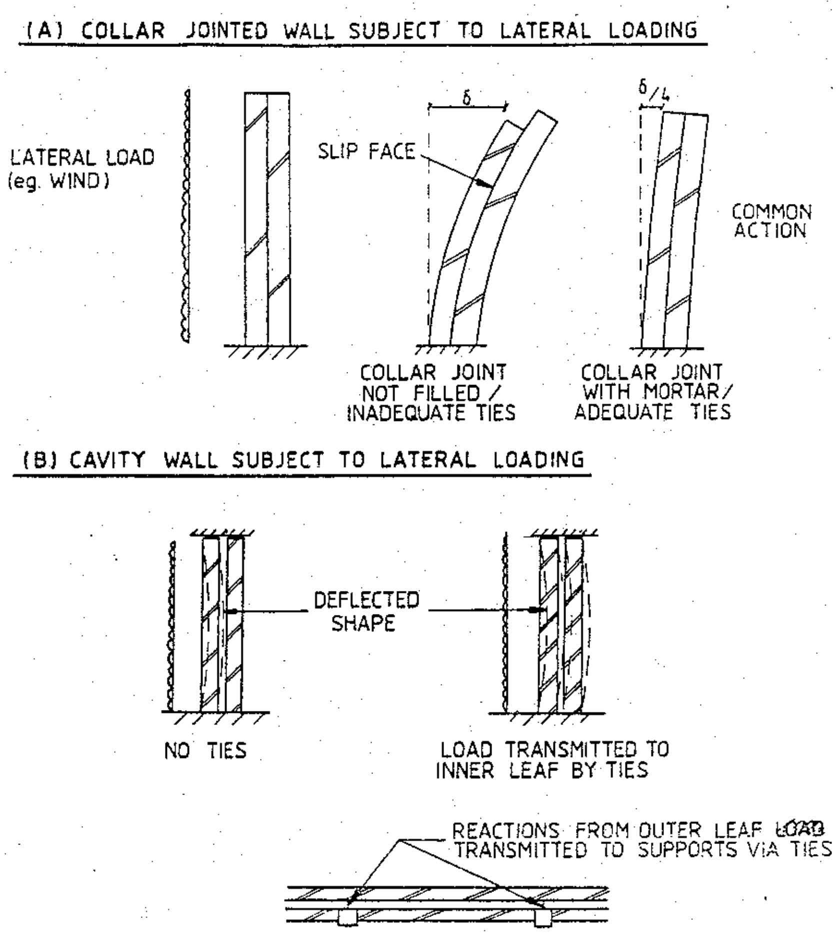


Figure 1.5: The function of wall ties

Table 4: Values for characteristic flexural strength of masonry, fkx, MPa

	Plane of failure parallel to bed joints			Plane of failure perpendicular to bed joints		
Mortar types	**M15 M20**	**M10 & M5**	**M2**	**M15 M20**	**M10 & M5**	**M2**
Clay units with a water absorption:						
less than 7%	0,7	0,5	0,4	2,0	1,5	1,2
between 7% and 12%	0,5	0,4	0,35	1,5	1,1	1,0
over 12%	0,4	0,3	0,25	1,1	0,9	0,8
Calcium silicate units		0,3	0,2		0,9	0,6
Concrete units or highly perforated units with a characteristic compressive strength $\geq$ 3.5 N/mm^2 used in walls of thickness:						
up to 100 mm		0,25	0,2		0,45	0,4
250 mm		0,15	0,1		0,25	0,2

1.7 Discrepancies with the National Building Regulations

SABS 0400 (11) contains deemed-to-satisfy rules for the structural design of unreinforced masonry structures. In some instances, particularly where walls are exposed to high wind loads or earth loads, the size of walls based on structural calculations in accordance with SABS 0164 Part 1 will not always be more economical than that which would have been obtained from use of these deemed-to-satisfy rules. This is due to inconsistencies and a lack of harmony between deemed-to-satisfy rules and structural codes of practice.

Examples of these inconsistencies are:

i) The permissable dimension of walls given in Table 2, Part K are often greater than that permitted in terms of Clause 5.5.2 of Part 1:

- 90 mm wall - 6,0 m c.f. 50 x 0,090 = 4,5 m
- 110 mm wall - 6,0 to 7,0 m c.f. 50 x 0,110 = 5,5 m
- 90 mm cavity wall - 5,0 to 8,0 c.f. 50 x 0,66.0,180 x 5,95 m
- 110 mm cavity wall - 6,0 to 9,0 m c.f. 50 x 0,66.0,22 = 7,33 m

ii) Table 2 of Part K does not restrict the size of openings and permits the tabulated wall dimensions to be applied to external cladding on buildings up to 25 m in height in any wind terrain. By comparison BS 5628 Part 3 (12), a code which offers rules for design based on structural calculations, tabulates maximum wall areas for walls in buildings of four storeys and less, situated in wind terrain categories 3 and 4, which have unsupported window openings less than 10% of the wall area.

iii) Part K does not differentiate between units of brick and block size with regard to the lateral strength of wall panels, free-standing walls and retaining walls. Table 4 of Part 1, on the other hand, indicates that the maximum flexural strength of a brick size unit can be up to twice that of a block size unit.

iv) Part K gives heights for free-standing walls without differentiating between wind terrain and unit types.

1.8 <u>Future amendments to SABS 0164 Part 1</u>

Future amendments to Part 1 should include:

* Changes in terminology to adequately reflect compatibility with SABS 0160 eg. characteristic loads should be referred to as nominal loads.

* Cross reference to the National Building Regulations and the code of practice for Masonry Walling which is in course of preparation.

* Reference to the load factors contained in SABS 0160.

* Changes in the tabulated values for characteristic shear strengths for class II mortar.

* A value for the characteristic shear strength in the vertical direction for brickwork.

* Formulae which more accurately reflect the shear distribution in non-rectangular wall sections and enable the shear stress at flange-rib/pier interfaces and at the collar joints in multi-leaf walls to be assessed.

* Values for the characteristic vertical shear strength.

* More comprehensive rules for the spacing of ties and headers in collar jointed walls.

* Amendments to reflect current improvements to BS 5628 Part 1.

* Stiffness coefficients for Z-shaped and diaphragm walls.

* Additional requirements relating to the strength and stiffness of cavity wall ties.

* Some quantitative guidance on the shear strength of damp proof courses.

Furthermore, the following areas should be reassessed:

* References to masonry units i.e. bricks and blocks.
* The values for characteristic flexural strengths.
* The use of class III mortar.

2. Part 2: Reinforced and prestressed masonry

2.1 Introduction

Masonry, like concrete, is strong in compression but very weak in tension i.e. their flexural tensile strength is often less than 5% of their compressive strength. However again like concrete, masonry can be reinforced to carry the tensile stresses or prestressed to eliminate them.

Both structural masonry codes are based on the limit state design philosophy. The unreinforced masonry code is based on ultimate limit states while the reinforced and prestressed code is based on ultimate and serviceability limit states.

With the ultimate limit state abrupt brittle failure at ultimate limit state can be expected while with reinforced masonry there is a warning of failure as ductile failure occurs. Unreinforced masonry is primarily concerned with compressive stresses while reinforced masonry is concerned with compressive and flexural stresses.

In Part 2 the serviceability limit state requirements are similar to those in the structural concrete code.

Part 2 is based on the British code of practice BS 5628 Part 2 with the following points of departure:

i) Higher partial factors of safety for material strength.
ii) Different partial factors of safety for loads (Factors contained in SABS 0160 have been adopted).
iii) Different exposure categories and levels of corrosion protection for steel reinforcement (Categories of exposure and levels of protection are based on AS 3700).

Reinforcement may either be placed in the cores of hollow units or in cavities created by masonry bonding patterns. SABS 0164 Part 2 refers to the following 4 types of reinforced masonry construction which are illustrated in Figure 2.1: (Page 2-2)

i) grouted-cavity masonry
ii) pocket-type masonry
iii) quetta bond masonry
iv) reinforced hollow blockwork.

Post-tensioning bars are also commonly placed in the cavities of diaphragm walls as illustrated in Figure 2.2.

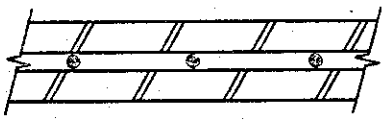
Methods of filling around the reinforcement must be considered. If grouting is used, care must be taken in the construction to ensure that the cavities are free of mortar droppings. Particular care should also be taken to:

i) Prevent air from being trapped below the grout/infill concrete.
ii) Adequately protect the reinforcement from corrosion.
iii) Ensure that the masonry can safely sustain the hydrostatic pressures exerted by the grout/infill concrete.

FIGURE 2.1 : TYPES OF REINFORCED MASONRY WALLS

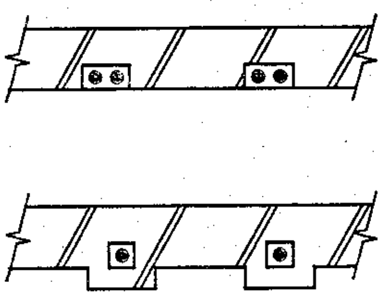
NOTES

1) GROUTED CAVITY



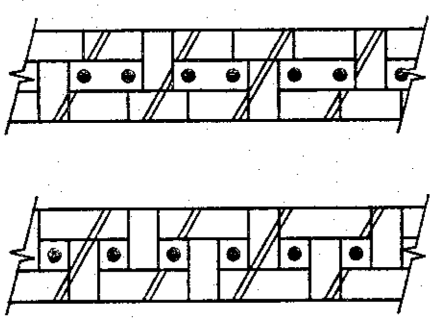
- Maximum bar diam = 25mm.
- 25 MPa (min.) concrete or mortar infill
- Secondary steel ≮ 0,05%
- Maximum bar spacing = 500mm.
- Minumum cover = 20mm.
- Galvanised reinforcement should be considered when mortar is used as infill
- Vertical twist ties to be installed for low lift grouting construction and more substantial ties for high lift grouting

2) POCKET TYPE



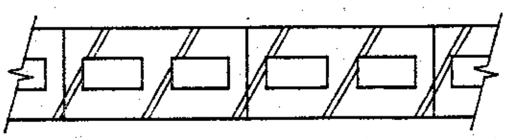
- Maximum bar diam = 32mm.
- 25 MPa (min) concrete infill
- No secondary reinforcement required
- Cover to steel dependant on exposure and grade of concrete
- Flange width ≯ rib width + 12 x flange width
  ≯ rib spacing
  ≯ 1/3 wall height

3) QUETTA BOND



- Mortar to be used as infill unless cavities are large
- Secondary reinforcement ≯ 6mm. diam
- Maximum bar spacing = 500mm.
- Maximum bar diam = 25mm.
- Secondary reinforcement ≮ 0,05%

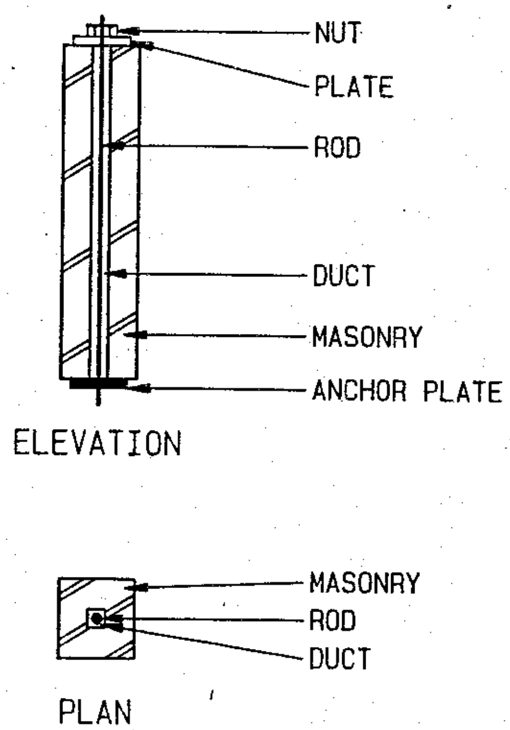
4) HOLLOW UNITS



- Shell bedding unsuitable
- Maximum bar diam = 25mm.
- Secondary reinforcement ≯ 6mm. diam
- Secondary reinforcement ≮ 0,05%
- Maximum bar spacing = 500mm.
- 25 MPa (min.) concrete infil or grout

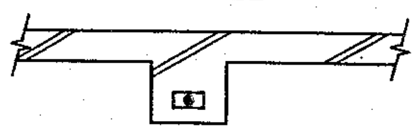
FIGURE 2.2 : TYPES OF PRESTRESSED MASONRY WALLS

1) ELEMENTS OF PRESTRESSING SYSTEM



2) POPULAR FORMS OF WALL CONSTRUCTION

FIN WALL



DIAPHRAGM WALLS

2.2 General comments on particular sections of the code

2.2.1 Definitions

In Part 2, the number of definitions have been extended from Part 1 and now cover masonry units of brick and block size, solid and hollow, the dimensions of a frog, and types of reinforced masonry. However, some of the definitions differ from those in SABS 1215[13] and SABS 0400[14]. Upper limits on the length and width of masonry units are stated, while limits for height are also stated.

In terms of Part 2, hollow masonry units may contain cavities in excess of 25% but not exceeding 60% of the gross volume of the unit. Formerly [13,14] 50% was the upper limit for cavities.

2.2.2 Materials and components

In Part 2, the section on materials and components has been extended to cover their use in the construction of reinforced and prestressed masonry. Because of the possibility of corrosion due to the presence of chlorides in the aggregate, the permissible amount of chlorides in sand and concrete is stated, as well as in admixtures.

Reinforced masonry may require the use of special units, unusual wall ties, and so on, which may not be commonly available and these need to be carefully described in the specification.

Fixing components, usually of metal, are used to connect structural masonry elements to structural elements in other materials. Typical examples are steel roof trusses and concrete floor and roof slab connections, restraining and tieing straps, shear transfer and horizontal restraint fixings.

2.2.3 Research carried out on reinforced masonry has led to the conclusion that reinforced concrete design principles can be applied, with appropriate modifications. Consequently, the design guidance given is in may instances almost identical to that given in CP110[(26)].

2.2.4 Other design considerations

Table 17 in SABS 0164-2 classifies the various surface and exposure conditions expected in South Africa together with a map of climatic zones.

This is a section which is concerned primarily with the durability of the structure. As such it covers masonry units and mortar (it refers to a new SABS code of practice at present in its final stages of completion entitled Masonry Walling Part 1 Guidelines for design and construction) and resistance of metal components to corrosion; joint finishing and fire resistance. Of concern is the possible corrosion of reinforcement in structural masonry.

The selection of reinforcement for durability is related to the corrosion-resistance rating for steel in masonry and the expected exposure conditions. The surface and exposure environment relate to surface of members in contact with the ground, interior and exterior environments, and water.

Table 17 classifies the various surface and exposure conditions expected in South Africa together with a map of climatic zones.

2.2.5 Work on site

In Part 1, aspects of work on site are included in an appendix. With reinforced and prestressed masonry where materials and components are more highly stressed, wall thicknesses are thinner and the structure generally less robust, the section on work on site in Part 2 forms an integral part of the code.

In many respects, the workmanship aspects of reinforced and prestressed masonry are the most critical in relation to the successful and widespread use as a structural material[5].

For practical reasons, reinforced masonry will generally be required to be more accurate than reinforced concrete, because the masonry will nearly always provide the architectural finish, whereas the concrete structure will usually have some applied finish.

Six forms of reinforced masonry wall construction are referred to in detail in Part 2 namely: grouted cavity construction, high and low lift; reinforced hollow blockwork, high and low lift; Quetta and similar.

The designer and detailer should avoid as far as possible the need for special-shaped masonry units, cutting masonry units, the use of difficult bonding (such as quetta bond) a mixture of units of differing strength, type and material and similar causes of site delays and problems. Simplicity of construction without sacrificing structural safety or efficiency - is important.

The construction control to be employed for all reinforced and prestressed masonry shall be category 1, i.e. the quality control of workmanship is maintained by:

a) either frequent visits to the site by the engineer or the presence of his permanent representative on site, to ensure that the work is built in accordance with the provisions of SABS 0164 and any such specifications as he may specify; or
b) preliminary and site sampling and testing.

2.2.6 Annexes

Annexes covering wall ties for high-lift cavity wall, design method for walls incorporating bed joint reinforcement to enhance lateral load resistance, estimation of deflection, and methods of determination of characteristic compressive strength of masonry units forms an integral part of the code.

PART 3
WORKED EXAMPLES
REINFORCED CONCRETE MASONRY TO SABS 0164 - PART 2: 1992

3.1 Basis of Design

3.1.1 Limit state approach
3.1.2 At ultimate - design strength ≥ design load
3.1.3 At service - final deflection:
≤ length/125 for cantilevers
≤ span/250 for other elements
3.1/4 Cracking - limit to size only

3.2 Strength of Materials

3.2.1 Characteristic compressive strength of masonry, f_k, from Table 4, SABS 0164, Part I.

3.2.2 For sections in which main reinforcement is placed within pockets, cores or cavities filled with concrete infill the following applies (the values are slightly less for reinforcement surrounded with mortar):

o Characteristic Shear Strength, f_v:

$$f_v = 0,35 + 17,5\left(\frac{A_s}{bd}\right), < 0,7 MPa$$

For simply supported reinforced beams or cantilevered retaining walls where $a/d \leq 6$, f_v obtained above may be increased by a factor

$$2,5 - 0,25\left(\frac{a}{d}\right), \leq 1,75 MPa$$

with a = shear span and d = effective depth
(shear span is the ratio of the maximum design bending moment to the maximum design shear force).

o Characteristic Bond Strength, f_b (in tension or in compression):
- Plain bars = 1,8 MPa
- Deformed bars = 2,5 MPa

3. The characteristic tensile strength of reinforcing steel, f_y is given in Table 7, but essentially:

o Mild steel, f_y = 250 MPa
o High yield steel, f_y = 450 MPa
o Hard drawn wire, f_y = 485 MPa

3.3 Partial Safety Factors

3.3.1 Loads

- o As dealt with in Parts 1 and 2 of the preceding lectures.
- o Materials

3.3.2 Material design strength = characteristic strength divided by the appropriate partial safety factor, γ_m

- o Masonry in compression or bending - γ_{mm}

  is either 2,3 or 2,6 for special (category A) or normal (category B) manufacturing control; special category site control is assumed for this type of construction.

  - Shear, γ_{mv} = 2,0
  - Bond, γ_{mb} = 1,4
  - Steel, γ_{ms} = 1,15

3.4 Design of Flexural Elements

3.4.1 General

- o Axial thrust < 0,1 $f_k.A_m$ may be ignored(i.e. section may be designed for pure bending) where A_m = cross sectional area of masonry)
- o Effective span (similar to reinforced concrete)
- o Simply supported or continuous members, smaller of:
  - c/c of supports
  - clear distance between supports + effective depth
- o Cantilever, smaller of:
  - end of cantilever to centre of support
  - end of cantilever to face of support + half effective depth

$$M_d < \frac{0,4.f_k.b.d^2}{\gamma_{mm}}$$

where

$$z = d(1 - \frac{0,5.A_s.f_y.\gamma_{mm}}{b.d.f_k.\gamma_{ms}}) \leq 0,95.d$$

o Flanged sections

$$M_d = \frac{A_s.f_y.z}{\gamma_{ms}}$$

but

$$M_d < \frac{f_k}{\gamma_{mm}}.b.t_f(d - 0,5t_f)$$

where flange thickness, t_f = thickness of masonry but ≤ 0,5d

- The flange width should be taken as the least of:

i) for pocket type walls, the width of pocket or rib + 12 t_f
ii) the spacing of the pocket or ribs
iii) one-third the height of the wall

3.4.4 Shear

o Shear stress, due to design load

$$v = \frac{V}{b.d}$$

o Span/effective depth ratios (similar to reinforced concrete)

	End Condition	Ratio
Laterally loaded walls	- Simply supported - Continuous or spanning in two directions - Cantilever with As $\leq$ 0,5%	35 45 18

	End Condition	Ratio
Beams	- Simply supported - Continuous - Cantilever	20 26 7

o Lateral stability (similar to reinforced concrete)

- Simply supported or continuous beams:

  Clear distance between lateral restraints $\leq$ lesser of 60 b_c or $250b_c^2/d$ where b_c = width of compression face between restraints

- Cantilever (lateral restraint only at support):

  Clear distance from end of cantilever to face of support $\leq$ lesser of 25 b_c or $100b_c^2/d$ where b_c = width of compression flange at support.

3.4.2 Assumptions

- o Plane sections remain plane
- o Rectangular stress block, width = f_k/γ_{mm}
- o Maximum strain in outermost compression fibre at failure = 0,0035
- o Tensile strength of masonry ignored
- o Stress in reinforcement from Table 7 and stress strain diagram Figure 1 of SABS 0164-2
- o Span/d $\geq$ 1,5

3.4.3 Design formulae

o Rectangular sections

$$M_d = \frac{A_s \cdot f_y \cdot z}{\gamma_{ms}}$$

but

- o Single leaf, t_{ef} = actual thickness
- o Grouted cavity wall
  - t_{ef} = overall thickness if cavity width $\leq$ 100 mm
  - t_{ef} = thickness of two leaves + 100 mm if cavity width > 100 mm
- o For cavity walls and for columns with only one leaf reinforced, t_{ef} to be taken as greater of
  - 2/3 sum of actual thickness of two leaves
  - thickness of thicker leaf.

3.6.6 Short columns

- o Slenderness $\leq$ 12
- o Generally design for the maximum moment about the critical axis only
- o Design using either:
  - basic assumptions
  - code formulae
  - charts
- o Using code formulae
  - If design actual load, N $\leq$ design actual resistance, N_d, only minimum reinforcement required

where

$$N_d = \frac{f_k}{\gamma_{mm}} . b(t-2e_x)$$

where t = overall thickness of section in plane of bending

$e_x = M/N \leq 0{,}5t$

- Where N > N_d given above, the section is assessed using the relation $f_{s1} = 0{,}83\ f_y$ and the following equations and if $v < f_v/\gamma_{mv}$ (with $\gamma_{mv} = 2{,}0$) no shear reinforcement required, but nominal links may be considered

- If $v > f_v/\gamma_{mv}$ shear reinforcement required
- Provide shear reinforcement according to

$$\frac{A_s}{s_v} \geq \frac{b\left(v - \frac{f_v}{\gamma_{mv}}\right)\gamma_{ms}}{f_y}$$

but $v \leq 2{,}0/\gamma_{mv}$ MPa

3.4.5 Deflection and cracking

Calculations not necessary if span/effective depth OK and detailing rules followed.

3.5 Design of Elements subjected to a combination of vertical loading and bending

3.5.1 General

- o Members subject to a substantial vertical and horizontal loading, or
- o members subject to eccentric vertical loads ($e_x > 0{,}05$ t).

3.5.2 Slenderness limits

- o Cantilever walls and columns $\leq$ 18
- o Other walls and columns $\leq$ 27

3.5.3 Lateral supports - simple and enhanced

Enhanced when any floor or roof span on to the wall from both sides at the same level or when in-situ concrete, reinforced masonry or precast concrete floor or roof giving equivalent restraint have a bearing of at least one half of the thickness of the wall onto which it spans, but not less than 90 mm.

3.5.4 Effective height, h_{ef}, of walls and columns

Table 13 gives the effective heights for various end conditions; this is similar to that for other materials.

3.5.5 Effective thickness, t_{ef}

$$N_d = \frac{f_k}{\gamma_{mm}} b d_c + \frac{f_{s1} A_{s1}}{\gamma_{ms}} - \frac{f_{s2} A_{s2}}{\gamma_{ms}}$$

$$M_d = 0{,}5 \frac{f_k}{\gamma_{mm}} b d_c (t - d_c) + \frac{f_{s1} A_{s1}}{\gamma_{ms}} (0{,}5t - d_1) + \frac{f_{s2} A_{s2}}{\gamma_{ms}} (0{,}5t - d_2)$$

A_{s2} = area of compressive reinforcement in most compressed face

A_{s2} = is considered as compressive, inactive or tensile depending on the resultant eccentricity of the load

f_{s1} = stress in reinforcement in most compressed face

f_{s2} = stress in reinforcement in least compressed face

= $-0{,}83 f_y$ in compression or $+ f_y$ in tension.

- A value for d_c is chosen which ensures $N_d > N$ and $M_d > M$. The choice of d_c establishes the assumed stress-strain distribution and stresses in the reinforcement can then be determined from the stress-strain relationship or as follows:

  i) $d_c = t$, then f_{s2} varies linearly between 0 and $-0{,}83\ f_y$
  ii) $(t - d_2) < d_c < t$, then $f_{s2} = 0$
  iii) $(t - d_2) > d_c > t/2$, then f_{s2} varies linearly between 0 and $+ f_y$
  iv) $t/2 > d_c > 2d_1$, then $f_{s2} = + f_y$
  v) $d_c \geq 2d_1$

- As an alternative to the aforementioned where $e_x > (t/2 - d_1)$, the axial load may be ignored and the section designed to resist an increased moment $M_a = M + N(t/2 - d_1)$, the area of tension reinforcement necessary to provide resistance to this increased moment may be reduced by

$$N\gamma_{ms}/f_y$$

- o Using Charts

  - For charts the following values can be calculated and depicted in graph form:

    $N/b.t.f_k$ and $M/b.t^2.f_k$, then for known f_y and d/t ratios find ρ_1/f_k where $\rho_1 = A_s/b.t$

3.5.7 Slender columns

- o Slenderness > 12
- o For bending about one axis only, design as above for short columns but add additional moment, M_a

$$M_a = \frac{N(h_{ef})^2}{2000t}$$

3.5.8 Short walls

- o Slenderness ratio $\leq$ 12
- o Design using basic assumption but if e_x (= M/N) $\geq$ 0,5t consider bending only

3.5.9 Slender Walls

- o Slenderness ratio > 12
- o Design as for short walls but include additional bending moment, M_a

3.6 **Elements subjected to axial compressive loading only**

- o Resultant eccentricity $\leq$ 0,05t
- o Design either

  - Clause 5.4 of SABS 0164: Part I (unreinforced)

$$N_d = \frac{\beta b t f_k}{\gamma_{mm}}$$

where ß is the capacity reduction factor as in Table 8 of SABS 0164-1, which accounts for the slenderness of the wall and the eccentricity of the load at the top of the wall (γ_{mm} in accordance with SABS 0164-1)

or

- Using the approach for reinforced columns described in SABS 0162 - Part 2, however this is unlikely to give a more favourable result and the solution is generally more onerous.

3.7 Detailing

3.7.1 Maximum size of reinforcement

o	in joints	= 6 mm
o	pocket-type walls	= 32 mm
o	elsewhere	= 25 mm

3.7.2 Minimum of secondary reinforcement (all steel types)

o 0,05% (breadth x effective depth)

3.7.3 Spacing of bars

o minimum = greater of

- max. aggregate size + 5 mm or
- bar size, but not less than 10mm

o maximum = 500 mm, except in cores and pockets

3.7.4 Beam links (if provided) - nominal shear reinforcement

o A_{sv}/S_v = 0,002 b_t or 0,0012 b_t for mild and high yield steel respectively, with b_t = breadth at reinforcement level.

$S_v \leq 0,75\ d$

3.7.5 Column links - consider need for links

o If $A_s > 0,25\%A_m$ and more than 25% of axial load capacity will be used, then provide links. If $A_s \leq 0,25\%A_m$ links need not be provided.

o Where links are provided:

- Size $\geq$ 6 mm

- Spacing $\leq$ lesser of:

  i) least lateral dimension of column
  ii) 50 x link diameter

iii) 20 x main bar diameter

3.7.6 Anchorage bond

- o Stress in reinforcement $\leq l.f_b.u/\gamma_{mb}.A_s$

  with l = anchorage length

3.7.7 Laps, hooks and bends, curtailment and anchorage

- o Guidance given in SABS 0164 Part 2, paragraph 7.6.8 and 7.6.9

3.7.8 Fire resistance

- o This is covered in SABS 0164 - Part 2, taking masonry as part of the cover.

3.7.9 Durability

- o Four types of exposure classifications are given, i.e. E1, E2, E3 and S with the corresponding cover to the steel. Features likely to be subjected to more severe exposure than remainder of building shall be given special attention

- o If burnt clay masonry units have a water absorption > 10% and hollow concrete masonry units have a density < 1500 kg/m^3 then steel with the next exposure situation should be used (or stainless steel if appropriate)

3.8 Prestressed Masonry

3.8.1 Design methods are given to ensure that the requirements for both ultimate and the serviceability limit states are satisfied

- o For serviceability limit state there is a maximum value for the compressive strength in the masonry, after losses have occurred

3.8.2 Assumptions

Essentially as for reinforced concrete with specific guidance on bonded and unbonded tendons

3.8.3 Prestressing tendons

- o Initial prestress ≤ 70 % of characteristic stress in tendon
- o Loss of prestress is similar to reinforced concrete

3.8.4 Detailing

- o Guidance is given for local bearing stress at anchorage
- o The tensioning sequence should be indicated to prevent overstressing
- o Links are provided as for reinforced masonry

Examples - flexural elements

Example 1:

Determine the characteristic compressive strength (f_k) of hollow concrete block masonry with nominal strength of 3,5 and 7 MPa, filled with Class 25 concrete in situ.

Characteristic compressive strengths for hollow blocks masonry with Class II mortar, SABS 0164:1 - 1980, Table 3(b),b states, that the characteristic compressive strengths, f_k of blocks shall be assessed on the net area of the block instead of the gross area. The characteristic strengths of the blocks are then determined as for a solid units. The block strengths based on the net area are all less than the infill concrete strength, therefore the f_k values are based on the block strengths throughout.

Block strength (based on net area)

$$= \frac{\text{Nominal compressive strength of block (MPa)}}{\text{(\% of solid material in hollow unit)}}$$

viz, for 3,5 MPa, 90 wide block the % of solid material in the hollow unit is 70% (see Note), therefore the block strength (based on the net area)

$$= 3,5/0,7 = 5,00 \text{ MPa} < 20 \text{ MPa}$$

Table 1: Characteristic Compressive Strengths, f^{2k} for hollow concrete blockwork, filled with 25 MPa infill concrete for Class II mortar.

		Characteristic Compressive strength based on nett area, MPa			
Height of Hollow Blocks	**% of solid material in hollow unit**	**(3,5) Block strength**	**fk**	**(7,0) Block strength**	**fk**
190 High					
90 Wide	70	5,00	4,457	10,00	7,800
140 Wide	55	6,36	4,313	12,73	7,288
190 Wide	51	6,86	3,650	13,73	6,458
140 High					
90 Wide	70	5,00	3,769	10,00	6,600
140 Wide	55	6,36	3,650	12,73	6,093
190 Wide	51	3,86	3,406	13,73	5,517^{+}

Note: The following figures comply with the requirements of SABS 1215:

Hollow unit width (mm)	% Solid material
190	51
140	55
90	70

Values interpolated from SABS 0164: Part I - 1980 for various aspect ratios:

+ Hollow block, 140 high and 190 wide -
h/w (aspect ratio) = 0,737;

Nominal compressive strengths for 7 Mpa hollow block based on net area
= 7/0,51 = 13,73 MPa

for 7 MPa: for 14 MPa:

$$\frac{fk-3.2}{0,737-0,6} = \frac{6,0-3,2}{2,0-0,6} \qquad \frac{fk-5,1}{0,737-0,6} = \frac{10,2-5,1}{2,0-0,6}$$

$$f_k = \frac{0,137.2,8}{1,4} + 3,2 \qquad f_k = \frac{0,137.5,1}{1,4} + 5,1$$

$$f_k = 3,474 \text{ MPa} \qquad f_k = 5,599 \text{ MPa}$$

for 13,73 MPa:

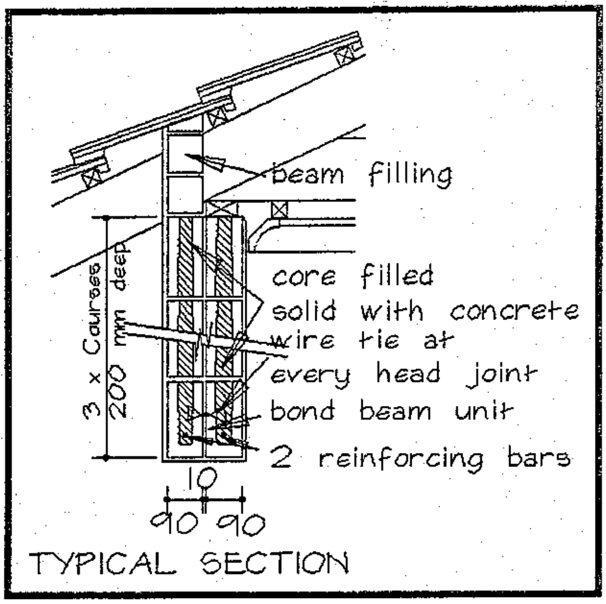
$$\frac{fk-3,474}{13,73-7,0} = \frac{5,599-3,474}{14,0-7,0}$$

$$f_k = \frac{6,730.2,125}{7,0} + 3,474$$

$$f_k = 5,517 \text{ MPa}$$

Example 2:

Design a beam to span a 4,0 m opening in a blockwork wall. The beam is 3 courses deep and supports a tiled roof with trusses at 0,76 m centres.



Blocks	:	390 x 190 x 90 wide, double leaf, of 3,5 MPa units with 25 MPa infill concrete. Voids in block = 30 %.
Mortar	:	Class II
Reinforcement	:	f_y = 250 MPa
General	:	- γ_{mm} = 2,6 - Exposure condition E1 - Horizontal web of hollow block = 30 mm

Solution:

- Loading

Summary of loading on roofs - SABS 0160-1989 (As amended 1990)

Table 2 gives the summary of the loading for the combinations of partial loading factors for tiles (concrete and clay) for a 30 degree roof slope and a 6m roof span. The simplified wind loading design procedure was used for a terrain category 3. Live load reduction was applied. The self load for the tiles and ceiling were taken as 510 and 100 N/m^{2}respectively.

Table 2: Loading at truss supports - truss spacing at 760 centres

Slope	Load (kN)	Loading
30 degrees	2,76	$1,5D_n$
	4,22	$1,2D_n+1,6L_n$
	3,69	$1,2D_n+1,3W_n$
	-0,40	$0,9D_n+1,3W_n$ Uplift
	4,32	$1,2D_n+0,5L_n+1,3W_n$
	0,78	$1,2D_n+0,5L_n-1,3W_n$

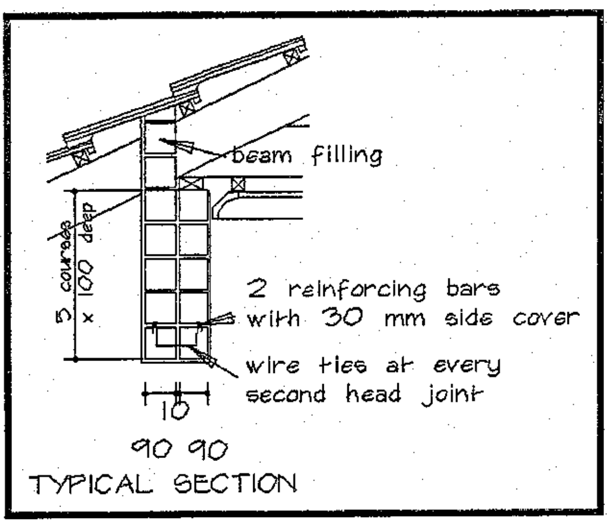
The maximum bending moment and shear force inclusive of the self weight of the 3 course lintel plus the beam filling is for 1,2Dn + 0,5Ln + 1,3Ln.

Input	Calculations	Output
	Maximum bending moment = 22,2 kN-m Maximum shear force = 19,3 kN	BM = 22,2 kN-m SF = 19,3 kN
SABS 0164:1 T 3(b)	f_k: For 3,5 MPa block with 70% solid, the net strength = 5 MPa, thus $f_k = 4{,}457$ MPa	$f_k = 4{,}457$ MPa
	d: Shell is 30 thick, cover is 20, say 12 mm dia. bar d = 3*course + 2*bedjoints - shell - cover - dia./2 = 3*190 + 2*10 - 30 - 20 -12/2 = 534 mm	d = 534 mm
	z: for 2 x R12 bars (one per block) $z = d(1 - 0{,}5.A_s.f_y.\gamma mm/b.d.f_k.\gamma_{ms})$ $= 534(1 - \dfrac{0{,}5.226.250.2{,}6}{90.2.534.4{,}457.1{,}15})$ = 454 mm	z = 454 mm
	$z/d = 454/534 = 0{,}85 \leq 0{,}95d$	$z/d < 0{,}95d$
	M_d: M_d = Moment of resistance based on reinforcement $= As.f_y.z/\gamma_{ms}$ = 226.250.454/1,15 = 22,3.106 N-mm = 22,3 kN-m > Design moment (BM=22,2 kN-m)	M_d=22,3 kN-m
	M_d = Moment of resistance based on masonry in bending compression $= 0{,}4.f_k.b.d^2/\gamma_{mm}$ $= 0{,}4.4{,}457.90.2.534^2/2{,}6$ = 35,2 kN-m > BM = 22,2 kN-m	M_d=35,2 kN-m
	Checks: Shear: v = SF/bd = 19,27.1000/90.2.534 = 0,20 MPa	v = 0,20 MPa
0164:2 6.2.4.1.2	$f_v = 0{,}35 + 17{,}5\rho$ = 0,35 + 17,5 x As/b.d = 0,35 + 226/2.90.534 = 0,39 MPa	

Input	Calculations	Output
SABS 0164:2 6.2.4.1.2	If shear span ratio, $a/d < 6$, i.e. $a = 22{,}2.1000/19{,}3 = 1150$ mm: $a/d = 1150/534 = 2{,}15 < 6$ therefore enhance f_v $f_v = 0{,}39.(2{,}5 - 0{,}25(a/d))$ $= 0{,}39.(2{,}5 - 0{,}25.2{,}15)$ $= 0{,}39.1{,}96$ $= 0{,}77$ MPa $f_v/\gamma_{mm} = 0{,}77/2{,}6 = 0{,}30$ MPa $f_v/\gamma_{mm} > v$ (0,20 MPa), therefore no shear reinforcement is required but nominal shear reinforcement can be considered.	f_v/γ_{mm} $= 0{,}30$ MPa Shear OK
	Local bond, f_{bs}:	f_{bs}/γ_{mb} $= 1{,}29$ MPa
6.2.7 T 10	$f_{bs}/\gamma_{mb} = 1{,}8/1{,}4 = 1{,}29$ MPa $SF/\Sigma_u.d = 19{,}3.1000/2.37{,}7.534$ $= 0{,}48$ MPa $f_{bs}/\gamma_{mb} > V/\Sigma_u.d$	f_{bs} OK
	Local Bearing: Assume 200 bearing length, V/bearing area = $\frac{19{,}3.1000}{20.90.2}$ $= 0{,}54$ MPa The bearing may be considered to be a bearing type 1 in SABS 0164-1 and therefore the characteristic bearing stresses may be increased by 25%, therefore $1{,}25.f_k/\gamma_{mm} = 1{,}25.4{,}457/2{,}6$ $= 2{,}14$ MPa $> 0{,}54$ MPa	local bearing OK
	Limiting Dimensions: Span/d = 3200/534 = 6 < 20	Span/d =OK
	Lateral Stability: 3200 < lesser of $60.b_c$ (10800) or $250.b_c^2/d$ (15169)	Lat Stab = OK
T12	Anchorage:	
7.2.3.3 7.6.9.2	12xdia. beyond centreline = 12x12 = 144 mm	

Example 3

Design a lintel to span a 2,0 m opening in a brickwork wall. The lintel is 5 courses deep and supports a tiled roof with trusses at 0,76 m centres. The beam is subjected to a moment of 5 kN-m and a shear force of 8,35 kN (including selfweight of lintel).



Units : 190 x 90 x 90, double leaf, of 14 MPa units.

Mortar : Class II

Reinforcement : f_y = 485 MPa - Hard drawn prestraightened wire with a minimum proof stress of 485 MPa as supplied by manufacturer of welded steel fabric reinforcement with 5,6 mm dia.

General :
- $\gamma_{mm} = 2,6$
- Exposure condition E1
- All bed and head joints are 10 mm thick.

Solution:

The maximum bending moment and shear force inclusive of the self weight of the 5 course lintel is for 1,2Dn + 0,5Ln + 1,3Ln.

Input	Calculations	Output
	Bending moment = 5,0 kN-m Shear force = 8,35 kN	BM = 5,0 kN-m SF = 8,35 kN
SABS 0164:1 T 3(a)	f_k: For 14 MPa bricks, f_k = 5,1 MPa	f_k = 5,1 MPa
	d: d =5*courses + 4*bedjoints - one course - bedjoint/2 = 5*90 + 4*10 - 90 - 10/2 = 395 mm	d = 395 mm
	As: $z = M_d\gamma_{ms}/A_s.f_y$ and $z = d(1 - 0,5.A_s.f_y.\gamma_{mm}/b.d.f_k.\gamma_{ms})$ therefore: $5.10^6.1,15/A_s.485 =$ $395\left(1 - \frac{0,5.A_s.485.2,6}{90.2.395.5,1.1,15}\right)$ $11856/A_s = 395 - 0,597\,A_s$ $11856 = 395.A_s - 0,597.A_s^2$ $A_s^2 - 662.A_s + 19859 = 0$ Hence, A_s = 32 or 631 mm², try 2 x Y5,6 bars: A_s = 49 mm²	2xY5,6(49)
	$z = 395\left(1 - \frac{0,5.49.485.2,6}{90.2.395.5,1.1,15}\right)$ = 366 mm	z = 366 mm
	z/d = 366/395 = 0,93 < 0,95	z/d < 0,95d
	Md = Moment of resistance based on reinforcement $= As.f_y.z/\gamma_{ms}$ = 49.485.366/1,15 = $7,56.10^6$ N-mm = 7,56 kN-m > Design moment (BM=5,0 kN-m)	M_d=7,56 kN-m
	M_d = Moment of resistance based on masonry in bending compression $= 0,4.f_k.b.d^2/\gamma_{mm}$ $= 0,4.5,1.90.2.395^2/2,6$ = 22,0 kN-m > BM = 5,0 kN-m	M_d=22,0 kN-m
0164:2 6.2.4.1.1	Checks: Shear:v = SF/bd = 8,35.1000/90.2.395 = 0,12 MPa f_v = 0,35 MPa	v = 0,12 MPa

Input	Calculations	Output
SABS 0164:2 6.2.4.1.2	If shear span ratio, $a/d < 2$, i.e. $a = 5,0.1000/8,35$ $= 599$ mm: $a/d = 599/395 = 1,52 < 2$ - therefore enhance f_v by $2d/av$ a_v = distance from face of support to principal load $= (2 - 2x0,76)/2$ $= 0,24$ m $= 240$ mm $f_v = 0,35.(2x395/240)$ $= 0,35.(3,29)$ $= 1,15$ MPa $= 0,70$ MPa not allowed $> 0,7$ MPa $f_v/\gamma_{mm} = 0,70/2,6 = 0,27$ MPa $f_v/\gamma_{mm} > v$ (0,12 MPa), therefore no shear reinforcement is required but nominal shear reinforcement can be considered -impractical !.	f_v/γ_{mm} $= 0,27$ MPa Shear OK
6.2.7 T 10	Local bond, f_{bs}: $f_{bs}/\gamma_{mb} = 1,5/1,4 = 1,07$ MPa $SF/\Sigma_u.d = 8,35.1000/2.17,59.395$ $= 0,60$ MPa $f_{bs}/\gamma_{mb} > V/\Sigma_u.d$	f_{bs}/γ_{mb} $= 1,07$ MPa f_{bs} OK
	Local Bearing: Assume 200 bearing length, V/bearing area $= \frac{8.35.1000}{200.90.2}$ $= 0,23$ MPa The bearing may be considered to be a bearing type 1 in SABS 0164-1 and therefore the characteristic bearing stresses may be increased by 25%. Therefore the bearing strength is: $1,25.f_k/\gamma_{mm} = 1,25.5,1/2,6$ $= 2,45$ MPa $> 0,23$ MPa	Local bearing OK
T12	Limiting Dimensions: Span/d $= 2200/395 = 6 < 20$	Span/d =OK
7.2.3.3 7.6.9.2	Lateral Stability: $3200 <$ lesser of $60.b_c$ (10800) or $250.b_c^2/d$ (20506)	Lat Stab = OK
	Anchorage: 12xdia. beyond centreline = $12x5,6 = 67$ mm	

Examples - Vertical loading

Example 4:

Determine the axial load capacity of a 2,8 m high wall and check that the cover to the steel is adequate.

Blocks : 390 x 190 x 190 wide, hollow, of unit strength 7 MPa with 51 % solid. Units filled with 25 MPa concrete.

Mortar : Class II

Reinforcement : One Y 12 bar in each core, f_y = 450 MPa, positioned in the centre of the blocks.

General :
- γ_{mm} = 2,6
- Exposure condition E1
- Horizontal web of hollow block = 30 mm
- Simple lateral support provided top and bottom

Solution:

SABS 0164 - 2

Table 18 Exposure condition E1 with 25 MPa infill concrete requires 20 mm cover.

Cover provided = 190/2 - 12/2 - 30
= 59 mm => adequate

Table 13 Simple lateral support provided,
therefore $h_{ef} = h = 2\ 800$

For single leaf wall, $t_{ef} = t = 190$

Slenderness ratio = 2800/190 = 14,7

From Table 8, SABS 0164 Part 1, ß = 0,87

For a 7 MPa block, 190 high and 190 wide, with 25 MPa infill concrete with 51 % solid, net strength = 13,73 MPa and f_k = 6,458 MPa

$$N_d = \beta b t f_k / \gamma_{mm}$$

$$= (0{,}87.1000.190.14{,}7)/2{,}6.1000$$

$$= 935 \text{ kN/m}$$

Example 5:

Design a 3,9 m high column, 390 x 390 with axial load of 300 kN and moment of 60 kN-m.

Blocks : 390 x 140 high x 190 wide, hollow, of unit strength 7 MPa with 51 % solid. Units filled with 25 MPa concrete.

Mortar : Class II

Reinforcement : Assume Y 25 bars, f_y = 450 MPa

General :
- γ_{mm} = 2,6
- Exposure condition E2
- Horizontal web of hollow block = 30 mm
- Lateral restraint in both directions top and bottom

Solution:

SABS 0164 - 2

Table 18 Exposure condition E2 with 30 MPa infill concrete requires 30 mm cover.

Cover provided = 190/2 - 25/2 - 30
= 53 mm => adequate

Table 13 Lateral support provided,

therefore h_{ef} = h = 3 900

For single leaf wall, t_{ef} = t = 190

Slenderness ratio = 3 900/390 = 10 < 12

=> short column

For a 7 MPa block, 140 high and 190 wide, with 30 MPa infill concrete with 51 % solid, net strength = 13,73 MPa and f_k = 5,517 MPa

Resultant eccentricity, e_x = 60/300 = 0,20 m = 20 mm

$N_d = (f_k/\gamma_{mm}).b.(t - 2ex)$

$= (5,517/2,6).390.(390 - 40).10^{-3}$

= 290 kN

The design axial load (300 kN) exceeds this, therefore a full analysis needs to be carried out.

$N_d = (f_k/\gamma_{mm}).b.d_c + (f_{s1}.A_{s1}/\gamma_{ms}) - (f_{s2}.A_{s2}/\gamma_{ms})$

It is now necessary to choose a value of d_c, which should not be chosen as less than $2d_1$, where d_1 is the depth from the surface to the reinforcement in the more highly compressed face which in this case is 95 mm, i.e. half of the block width if the reinforcement is placed in the core of the hollow block.

Choose d_c = 250 mm

This value is between $(t - d_2) = 390 - 95 = 295$, and $t/2 = 195$ (where d_2 is the depth to the reinforcement from the least compressed face). In this range, f_{s2} varies linearly between 0 and f_y, i.e. at 295 mm, $f_y = 0$ and at 195 mm, $f_{s2} = f_y$.

Thus for d_c = 250 mm:

$$f_{s2} = ((295 - 250)/(295 - 195)).f_y = 0{,}45\ f_y$$

$$N_d = (5{,}517/2{,}6x10^3).390.250 + (0{,}83.450.982/1{,}15x10^3)-(0{,}52.450.982/1{,}15x10^3)$$

$$= 326 \text{ kN} > 300 \text{ kN}$$

$$M_d = (0{,}5f_k/\gamma_{mm}).b.d_c(t - d_c) + (f_{s1}.A_{s1}/\gamma_{ms}).(0{,}5t - d_{1)} + (f_{s2}.A_{s2}/\gamma_{ms}).(0{,}5t - d_{2)}$$

$$M_d = (0{,}5.5{,}517/2{,}6.10^6).390.250.(390 - 250) + (0{,}83.450.982/1{,}15.10^6).(0{,}5.390 - 95) + (0{,}52.450.982/1{,}15.10^6).(0{,}5.390 - 95)$$

$$= 66 \text{ kN-m} > 60 \text{ kN-m}$$

Thus use 4xY25 bars, in each core.

REFERENCES

1. SOUTH AFRICAN BUREAU OF STANDARDS. Structural use of masonry. Part 1 - Unreinforced masonry walling. SABS 0164 Part 1 - 1980.

2. BRITISH STANDARDS INSTITUTIONS. Code of practice for structural use of masonry. Part 1 - Unreinforced masonry. BS 5628 : Part 1 : 1979.

3. SOUTH AFRICAN BUREAU OF STANDARDS. Code of practice for general procedures and loadings to be adopted in the design of buildings. SABS 0160-1989.

4. ACI- ASCE STANDARDS. Building code requirements for masonry Structures. ACI 530-88/ASCE 5-88.

5. HASELTINE, BA and MOORE, JFA. Handbook to BS 5628 : Structural use of masonry : Part 1 : Unreinforced masonry. The Brick Development Association, May 1981.

6. WATERMEYER, RB. The design of laterally loaded unreinforced walls. Lecture to Durban Branch of SAICE, Durban June 1989.

7. ROBERTS, JJ. Design of diaphragm walls, Structural Masonry. Lecture Series, Concrete Masonry Association, September 1991.

8. SOUTH AFRICAN BUREAU OF STANDARDS. Metal ties for Cavity Walls. SABS 28-1982.

9. STANDARDS AUSTRALIA. Wall ties for Masonry Construction. AS 2699.

10. STANDARDS AUSTRALIA. SAA Masonry Code AS. 3700-1988.

11. SOUTH AFRICAN BUREAU OF STANDARDS. The application of the National Building Regulations. SABS 0400 - 1990.

12. BRITISH STANDARDS INSTITUTION. Code of practice for the use of Masonry : Part 3 : Materials and components, design and workmanship BS 5628 Part 3.

13. SOUTH AFRICAN BUREAU OF STANDARDS. Standard specification Concrete masonry units. SABS 1215.

14. SOUTH AFRICAN BUREAU OF STANDARDS. Code of practice for the application of the National Building Regulations. SABS 0400.

15. WEST. HWH., HODGKINSON HR. and HASELTINE BA. Experimental Methods and Results of Tests on Small Specimens and Full sized Walls. The Structural Engineer, Vol 55, No. 10, Oct 1977.

16. HASELTINE BA., and TUTT JN. Implications of Research on Design Recommendations. The Structural Engineer, Vol. 64A No. 11, Nov. 1986.

17. ROBERTS JJ., TOVEY AK., CRANSTON WB. and BEEBY AW. Concrete Masonry Designer's Handbook. Eyre and Spotliswoelde Publications Ltd, 1983.

18. WEST HWH., HODGKINSON HE., HASELTINE BA. and DE VEKEY RC. Research results on Brickwork and Aggregate Blockwork since 1977. Structural Engineer, Vol. 64A No. 11, Nov 1986.

19. HASELTINE BA., WEST HWH and TUTT JN. Design of Walls to Resist Lateral Loads. The Structural Engineer, Vol 55, No. 10, Oct 1977.

20. SINHA BP. A Simplified Ultimate Load Analysis of Laterally Loaded Model Orthotropic Brickwork Panels of Low Tensile Strength. The Structural Engineer, Vol 56B, No. 4, Dec 1978.

21. HENDRY AW., SINHA BP and DAVIES SR. An introduction to Load Bearing Brickwork Design. John Wiley and Sons.

22. LOVEGROVE R. A discussion of 'Yield Lines' in Unreinforced Masonry. The Structural Engineer, Vol 66, No. 22, Nov 1988.

23. LAWRENCE JJ. Discussion on Lovegrove R: A discussion of 'Yield lines" in Unreinforced Masonry. The Structural Engineer, Vol 67, No. 8, April 1989.

24. NEW SOUTH WALES GOVERNMENT PUBLIC WORKS DEPARTMENT AND THE ASSOCIATION OF CONSULTING STRUCTURAL ENGINEERS OF NEW SOUTH WALES. Masonry code of practice. Assoe CSE of NSE 1984.

25. STANDARDS AUSTRALIA. Concrete Masonry units. AS 2733 - 1984.

26. BRITISH STANDARD INSTITUTION. The Structural use of concrete. CP110 - 1972.