

MASONRY MATERIALS AND DESIGN FOR MOVEMENT

*SAICE LECTURE COURSE
STRUCTURAL DIVISION
28 August 1991*

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Synopsis

An understanding of structural masonry requires a knowledge of the materials being used and their response to service loads and environmental factors.

This paper covers the main structural masonry materials, viz. burnt clay, calcium silicate and concrete masonry units, and mortar and their performance when used in masonry structures subject to imposed actions.

1.0 Introduction

Structural masonry has been a traditional form of construction over many millennia. Most masonry structures have been designed by rule-of-thumb or by reference to tables of empirical design. Few buildings have collapsed - many have cracked.

Developments in design have permitted the resistance of structural masonry to loading to be calculated with greater accuracy - the "material" can be used with greater confidence. However, serviceability limit states require that the deflection and deformation characteristics be considered in design.

2.0 Definitions

Masonry is an assemblage of masonry units joined together with mortar or grout. Masonry units may be of brick or block size, solid or hollow and composed of materials such as burnt clay, calcium silicate and concrete.

A block is any masonry unit which has a length of more than 300mm or a width of more than 130mm⁽¹⁾.

A brick is any masonry unit which is not a block⁽¹⁾.

(In SABS 0164⁽⁵⁾ there is no definition of brick and block sizes. In determining characteristic strength values for walls reference is made to the aspect ratio. SABS 0164 is based on British practice where a brick is defined as a masonry unit not exceeding 337,5mm in length, 225mm in thickness (width) or 112,5mm in height. A block is a masonry unit which when used in its normal aspect exceeds the length or width or height specified for bricks.

CEN (the Europeans Standards body for both the Community and EFTA countries) is at present studying the significance of unit size and aspect ratio on compressive strength and behaviour of units in a wall).

A Hollow unit

- a) contains at least one large hole or cavity of such size that the solid material in the unit constitutes between 50% and 75% of the total volume of the unit calculated from its overall dimensions, or
- b) when used in a wall (or in a leaf of a cavity wall), forms internal cavities that have a total area, in a horizontal plane, of more than 25% of the horizontal cross-sectional area of the wall (or leaf of a cavity wall)⁽²⁾.

3.0. Masonry Units

SABS standard specification SABS 227⁽³⁾,
285⁽⁴⁾ and 1215⁽²⁾ cover masonry units
composed of burnt clay, calcium silicate and concrete
respectively.

A summary of the requirements of these standard specifications is given in Appendix A.

The abovementioned SABS specifications do not provide sufficient technical data for an estimation of movement in walls, in which the units are used, to be made. Masonry units of different material respond differently to changes in temperature, moisture and stress. (See Sections 5).

4.0 Mortar

4.1 Introduction

The cost and quality of masonry work is significantly affected by the mortar used. Mortars may account for as little as 7% of the volume of the walls, but the role it plays and the influence it has on performance are far greater than the proportion indicates.

4.2 Functions

Mortar provides a bed for laying; bonds units together to give compressive and flexural strength to wall; and seals joint against rain penetration.

4.3 Composition

Four types of building mortar are detailed in SABS specification ⁽⁵⁾⁽⁶⁾.

Portland cement : sand
Portland cement : lime : sand
Portland cement : sand plus mortar plasticizer
Masonry cement : sand

The approximate limiting proportions of these mortars are detailed in Table 1.

Table 1. Proportions for mortar⁽⁵⁾

Mortar Class	Portland Cement kg	Lime litres	Sand measured loose and damp litres max	Masonry cement : Sand or Portland cement with mortar plasticizer kg : Sand litres, max
I	50	0-10	100	50 : 100
II	50	0-40	200	50 : 170
III	50	0-80	300	50 : 200

In general terms the purposes for which the various classes of mortar may be used are as follows⁽⁵⁾:

Class I: Highly stressed masonry incorporating high strength masonry units such as might be used in multi-storey loadbearing buildings and/or where highly aggressive environmental conditions expected.

Class II: Normal loadbearing applications, as well as parapets, balustrades, retaining structures, free-standing and garden walls and other walls potentially exposed to severe damp conditions.

Class II: Lightly stressed (e.g. single storey) bearing walls where exposure to dampness is not severe.

The compressive strength requirements for mortar are given in Table 2.⁽⁵⁾

Table 2: Requirements for mortar

Mortar Class	Compressive Strength at 28d, MPa, min	
	Preliminary (lab) tests	Work tests
I	14,5	10
II	7	5
III	2	1,5

4.4 Materials

4.4.1 Cementitious materials

Ordinary portland cement⁽⁷⁾ and portland cement 15, may be used in mortar. It is not advisable to use PBFC⁽⁸⁾. Unless the mortar sands are good quality (see item 4.4.3) 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⁽⁹⁾ or a mortar plasticizer⁽¹⁰⁾.

4.4.2 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 mortar drying out with the consequential effect of incomplete hydration of the portland cement.

Lime for mortar means hydrated lime, i.e. commercial bedding lime, and not quicklime or agricultural lime. It gives the best results when used with coarse sand rather than with clayey sands. Lime complying with class A2P of SABS 523 is most suitable. Lime should not be used with masonry cement.

4.4.3 Sand

Sand for mortar should comply with SABS 1090⁽¹¹⁾ and be well graded from 5mm downwards, in accordance with Table 1 therein (see Table 3).

Table 3: Grading requirements of sands for mortar (Extract from SABS 1090 Sand for plaster and mortar - Table)

B.S. Sieve (m)	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 which does not comply with SABS 1090 may be used only with the written consent of the specifier. It is recommended that the contractor obtains a report from a recognized concrete testing laboratory certifying whether or not the sand is suitable.

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, though contributing to workability, frequently leads to excessive shrinkage and cracking of the joints. Sands containing high percentage of clay tend to give a conveniently plastic mix, but also lead to undue shrinkage.

In the Portland Cement Institute publication, Introduction to Concrete and Mortar, details are given of a field test for quality of sand for mortar or plaster. The test is as follows: Provided that a sand yields a smooth, plastic and cohesive mix, its quality, based on "water demand", can be determined by the following test:

The quantities used should be weighed out on a kitchen scale which is in good order, and the test should be carried out on a smooth impervious surface. It is also important that the sample used is fairly representative of the bulk supply.

Procedure

1. Dry out a wheelbarrow full of the sand to be tested.
2. Weigh out 5kg cement, 25kg of the dry sand Measure out, each into a separate vessel, 5 litres, 1 litre and 1,5 litres water.
3. Mix the cement and sand dry 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"

A "good" or "average" sand should be used "for mortar in walling below dampproof course".

4.4.4 Mortar plasticizers/admixtures

Mortar plasticizers exercise a desirable effect on the workability or plasticity of the mortar in which they are used. Generally these admixtures have no effect on setting time (i.e. accelerate or retard the mortar setting) but may cause air-entrainment.

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, temperature of the mortar, amount of plasticizer used and other factors or job conditions.

Some ready mixed mortars, batched and mixed at a central location, contain a special set-controlling (extended life) admixture that keeps the mortar plastic and workable for a period of more than 2,5 hours, usually 24 to 36 hours, but up to 72 hours with certain admixtures.

4.4.5 Pigments

Pigments⁽¹²⁾ may be used to colour mortar, the dosage being dependent on the specific colour required. The recommended limitation on mineral oxide content is 7% of cement content.

The addition of a pigment to a mortar mix increases its water requirement. This may adversely affect the strength and drying shrinkage of the mortar or the workability of the mortar if extra water is not added to the mix.

4.5 Desirable properties of mortar

Important properties of mortar affecting the quality of masonry work are workability, water retention, compressive strength, bond strength, ability to accommodate movement and rate of strength development.

Workability is the property which allows mortar to be spread easily over the masonry unit and affects the satisfactory performance of the mortar and the productivity of the artisan.

Water retention is the measure of a mortar's resistance to the loss of water by the suction of a porous masonry unit. Good water retention properties are important to ensure that:

- water is prevented from bleeding out of the mortar;
- the mortar bed is prevented from stiffening too much and becoming unworkable before the unit can be placed in position;
- sufficient water is retained in the mortar to ensure proper hydration of the cement; and
- mortars will remain workable for a long time after they have been spread on bed joints. (This assists laying, proper bedding of the units and compaction of the joint by tooling).

Except for highly stressed structural masonry, the compressive strength of mortar is not a particularly important property. It has comparatively little influence on wall strength. (Compressive strength is measured by testing 100mm cubes of mortar. In practice the mortar bedding thickness is between 8 and 12mm. Mortar in this situation has a strength of 2 to 3 times that of an equivalent cube, i.e. a Class II mortar cube of 5 MPa compressive strength has a 10-15 MPa bedding mortar strength). However, as it is easier to measure, it tends to be used as a control criterion. Bond strength is a much more important property because of its effect on both the tensile and transverse strength of walls.

No reliable relationship exists between compressive and tensile strength of mortar, while bond strength (adhesion between mortar and masonry unit) bears even less relation to compressive strength unless suction and the plastic properties of the mortar are at or near the optimum conditions. The use of excessively strong mortars is undesirable except where the durability of the masonry work is important.

Bond strength is important not only in relation to compressive or shear strength of the masonry, but also in relation to the passage of moisture. Rain usually penetrates a wall through fine cracks between the structural units and the mortar, and only rarely through the body of the structural units or the mortar. The greater the tensile strength of the bond, the greater is the possibility that leakage is reduced. However, perpend joints incorrectly formed and tooled are passages for moisture penetration. Design and workmanship affect water permeability far more than materials do⁽¹⁷⁾.

Bond depends, to a marked degree, on the balance between the initial rate of absorption (suction) of the unit and the water retention properties of the mortar.

Clay masonry units with a high initial rate of absorption may require wetting to reduce absorption of water from the mortar to ensure that sufficient water remains in the mortar for proper hydration of the cement. Structural units of clay having an initial rate of absorption exceeding $1,8\text{kg/m}^2\cdot\text{min}$ should be wetted prior to laying, so as to reduce the rate to between 0,7 and $1,8\text{kg/m}^2\cdot\text{min}^{(5)}$. Where wetting is required, it should take place on the day prior to laying to obtain uniformity of moisture distribution throughout stockpile packs.

Calcium silicate and concrete masonry units should not be wetted unless otherwise approved. Mortar mixes containing lime give best bond with calcium silicate units with well graded sand complying with the requirements of SABS 1090. The best performance of concrete masonry, including maximum bond and lowest shrinkage, is achieved by using concrete masonry units which are as dry as relative humidity conditions permit. The addition of moisture to the concrete masonry units causes the units to expand. Upon drying, this added water could cause the shrinkage of the masonry. Consequently, concrete masonry units should not be wetted before laying.

The rate of loss of workability of mortar is significant. Too rapid a loss makes unit placing difficult. Too slow a loss reduces productivity of laying due to units floating in mortar and not giving rigidity to the newly laid wall for further laying of units.

Excessive retempering of mortar reduces mortar strength and increases shrinkage.

Mortars shrink on drying out and the amount of drying shrinkage is directly related to the water requirement of the mortar sand.

4.6 Testing

Where mortar is used in structural masonry, reference should be made to SABS 0164⁽⁵⁾. Category 1 construction control of SABS 0164 requires:

- i) laboratory determination of the strength of mortar of each class proposed for use on the project, using materials from the sources from which the site will be supplied, at least 6 weeks before the commencement of construction work on site.
- ii) regular testing of the mortar used on site for compressive strength, three test cubes being prepared for every 150m^2 of wall in which one grade of mortar is used or for every storey of the building, whichever gives the greater frequency of testing.

Category II construction control specifies no tests but good practice requires a sieve analysis of the sand at the commencement of the work and when changes are noted or once every 100m³. There should be daily visual checks on possible contamination of the sand. Reference should be made to SABS 1090⁽¹¹⁾ which gives requirements for sand related to grading, dust content, content of material of low density, organic impurities, freedom from sugar and soluble deleterious impurities.

Frequent visual checks of the consistency of the mortar should be made to ensure that it is appropriate for the suction of the masonry unit, rate of laying and vertical progress.

4.7 Workmanship

Investigations in Australia⁽¹⁸⁾ have shown that unsupervised site brickwork has only 55 to 62% of the strength of supervised brickwork.

The quality of workmanship also has a significant effect on movement and durability. Some important factors which relate to mortar are:

- use of unsatisfactory materials particularly mortar sands,
- incorrect proportioning and mixing of mortar,
- excessive retempering of mortar,
- incorrect jointing procedure,
- disturbance of units after laying, and
- failure to protect work from the weather.

5.0 Determination of movement in response to the environment

5.1 Introduction

In this section movement in structures is primarily related to horizontal and vertical lengthening or shortening of the structure due to environmental factors.

The determination of movement is a complex problem and not merely a summation or subtraction of extreme or individual values of thermal and moisture movement, creep, and so on. For example as a material expands due to increase in temperature, it may also shrink as moisture is lost⁽¹³⁾. Also each movement is controlled to some extent by the degree of restraint to which masonry is subjected. Furthermore, walls under high vertical stress move less horizontally than walls subjected to lower stress. At the same time, not all movements are reversible since severe contractions may induce cracking in the perpendicular joints as the mortar-unit's tensile strength is exceeded, or even splitting of the unit themselves if the mortar is stronger than the unit.

"Any estimation of movement has to rely to a great extent on engineering judgement, since many factors, such as the temperature and moisture content of the material at the time of construction, weather conditions and degree of restraint, are unpredictable"⁽¹⁴⁾.

Burnt clay and concrete masonry units respond differently to environmental conditions due to their different moisture, thermal, elastic and plastic flow (creep) properties.

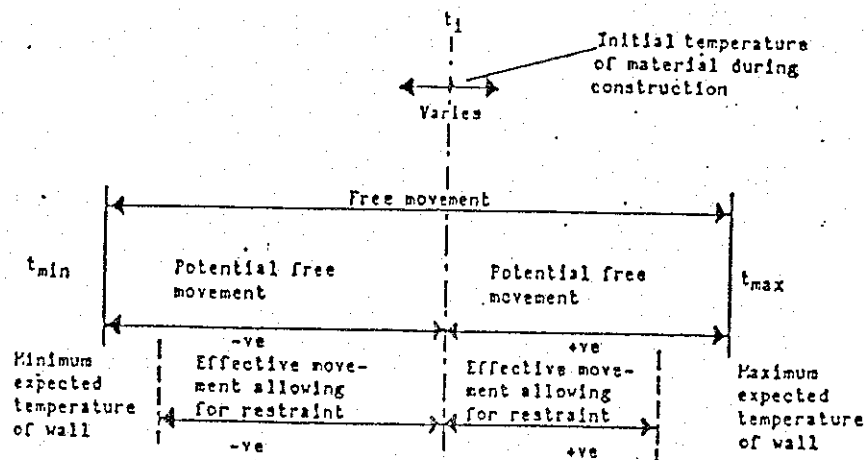
5.2 Factors causing movement

After construction, buildings are subject to small dimensional changes which may be caused by one or more of the following factors:

5.2.1 Changes in temperature

The total range of free movement due to thermal effect, which is generally reversible is equal to the temperature range multiplied by the appropriate coefficient of thermal expansion (see Figure 1). However, the movement that actually occurs within a wall after construction depends not only on the range of temperature but also on the initial temperature of the units as laid, on their moisture content, and the degree of restraint. To determine the effective free movement that could occur, therefore, some estimation of the initial temperature and temperature range must be made. The effective free movement so calculated must still be modified to allow for the effects of restraints.

Figure 1: Factors affecting thermal movement



Typical ranges of coefficients of thermal movement are given in Table 4⁽¹⁴⁾.

"The difference between mortars and units can largely be neglected when considering movement along the wall, since the effect of such differences will be controlled by the adhesion of the mortar to the units."⁽¹³⁾

Table 4: Linear thermal movement of masonry units and mortar

Material	Coefficient of linear thermal movement per °C $\times 10^{-6}$	mm per 10 m wall for 50° temperature change
Burnt clay masonry units (see note 1)	4-8	2-4
Concrete masonry units (see note 2)	7-14	3,5-7
Calcium silicate masonry units	11-15	5,5-7,5
Mortars	11-13	5,5-6,5

Note 1: Thermal movement of burnt clay masonry units depends on the clay mixture and its firing.

Note 2: Thermal movement of concrete masonry units depends on the type of aggregate and the mix proportions.

5.2.2 Changes in moisture content

5.2.2.1 Moisture expansion

Burnt clay units undergo an irreversible moisture expansion which occurs as a result of absorbing (i.e. the bonding of water molecules to the molecules of the masonry material as opposed to the entry of water molecules into the pores of the masonry) moisture from the atmosphere after firing. This expansion, which is characteristic of all porous ceramic products, commences once the unit starts absorbing moisture from the atmosphere - hence the term "moisture expansion".

SABS 227-1986⁽³⁾ contains a test procedure to assess the magnitude of moisture expansion that may take place in

units. However, this standard sets no limits for this parameter and suggests that limits should be agreed upon between the supplier and the purchaser. Nevertheless, the Industry has generally accepted the categorisation of clay units according to average moisture expansions based on an amendment to SABS 227-1970, published in 1980, which sets the following limits for the 96 hour standard steam test:

Category I	0,00% to 0,05%
Category II	0.05% to 0,10%
Category III	0,10% to 0,20%

(Although SABS 227-1986 expresses the results of a sample as an average, this average should be regarded as a design value since it includes the standard deviation of the results in its derivation).

Experience and research has revealed that: (18, 47)

- i) Units isolated from moisture will cease to expand.
- ii) Initially, expansion is rapid. However, the rate of expansion decreases with time. In most cases 50% of the expansion takes place in the first 6 months after coming out of the kiln, although, under coastal conditions, this may take place within 1 week. For all practical purposes, expansion is complete within 5 years.
- iii) A faster rate of expansion can be expected in hot humid climates and little or no expansion takes place in cold dry weather.
- iv) The expansion of burnt clay units is dependant on the specific clay minerals present in the product and the firing temperature of the process.
- v) Full scale tests have shown that irreversible movement in walls is approximately 66% of the movement of a single unit. There is more movement in the horizontal direction than the vertical direction, probably due to the lower ratio between units and joint material.
- vi) The problems associated with moisture expansion have been highlighted in times of high construction activity and shortages of supply during which

units are moved from the kiln to the building site with little or no stockpiling time period.

Except where crushed burnt clay bricks have been used as aggregates, concrete masonry units are not subject to irreversible moisture expansion.

Moisture expansion, if not catered for, can cause: (47)

- i) Cracking of reinforced concrete frame members where masonry infill panels abut hard up against the concrete.
- ii) Bowing of restrained panels where the support is eccentric to the panel.
- iii) Walls (particularly parapet walls) above the dpc oversailing the lower walls or supports.
- iv) Localised crushing of units.
- v) Longitudinal cracking and lifting of walls. (Frequently the roller course at the top of parapet walls lifts from the main body of the wall and bows in the vertical plane).
- vi) Cracking of walls at corners resulting in stability problems.
- vii) Jamming of doors, buckling of window sections and even cracking of window panes.
- viii) Rotation of short wall offsets.

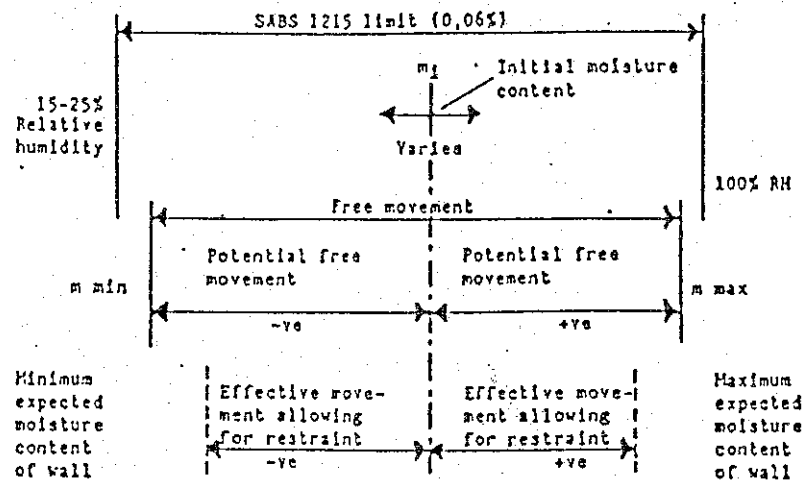
5.2.2.2 Reversible moisture movement

Burnt clay masonry units exhibit little movement with changes in moisture content; commonly not exceeding 1mm/10mm length and rarely exceeds 2mm/10m length.

SABS 285⁽⁴⁾ (calcium silicate) specifies a maximum drying shrinkage of 0,045% i.e. 4,5mm/10m wall length while SABS 1215⁽²⁾ (concrete) specifies a maximum drying shrinkage of 0,06% (shrinkage value based on difference in length between saturated and 15 to 25% relative humidity), i.e. 6mm/10m wall length. Thus the free movement may be expected to be less than the movement encountered during a drying shrinkage test, the amount being dependant on the extent of drying shrinkage that has taken place before the unit has been built in. (See

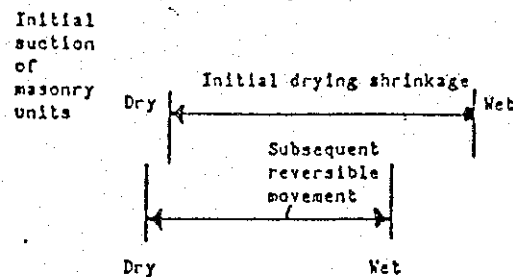
Figure 2). Reversible movements in a wall if unrestricted due to moisture changes are approximately 20 to 30% of the movement in the wall due to drying shrinkage i.e. 1,2 to 1,8mm/10m wall length.

Figure 2: Factors affecting moisture movement



The free moisture movement of mortar is similar to concrete masonry units. The initial drying shrinkage of mortars is generally between 0,04 to 0,10% i.e. 4 to 10mm/10mm wall length, but shrinkage of some South African mortars using clayey sand has been measured up to 0,34% i.e. 34mm/10m wall length⁽¹⁵⁾. The subsequent reversible movement is between 0,03 and 0,06% i.e. 3-6mm/10mm wall length⁽¹⁴⁾ (see Figure 3).

Figure 3: Factors affecting moisture movement of mortars



The moisture content of walls whose surfaces are rendered, plastered and/or painted is relatively constant and moisture movement is not a significant factor.

5.2.3 Carbonation

An additional shrinkage of concrete masonry units and mortar can occur as a result of carbonation of the hydrated portland cement by atmospheric carbon dioxide. The extent of carbonation and the subsequent movement depends on the permeability of the units and on the relative humidity. In dense units and in autoclaved units, carbonation shrinkage may be neglected since it is extremely small. In unprotected open textured units and mortar, the shrinkage due to carbonation may be between 20% and 30% of the initial free moisture movement i.e. between 1 and 2mm/10m wall length⁽¹⁴⁾.

5.3 Use of dissimilar materials in the same wall

Clay, concrete or calcium silicate units, because of their dissimilar movement tendencies should not be used in single leaf walls unless the different materials are separated by vertical control joints or horizontal damp proof courses or movement joints. The individual leaves of cavity walls may be constructed of dissimilar materials provided, however, that the interconnecting walls ties are flexible and reveals and wall ends are not in direct contact with each other.

Particular attention must be paid in loadbearing walls of cavity construction with dissimilar materials. Failures have occurred where cavity walls with a clay outer skin and concrete inner skin, support concrete floors. Initially, the load in each leaf may be proportioned according to the stiffness and area of each leaf. In time, the clay leaf may expand whilst the concrete leaf may shrink leading to a situation where all the load might be carried by the clay leaf, a load substantially higher than which it was designed for. This may lead to structural failure.⁽⁴⁶⁾

5.4 Estimation of total movement within a wall

"To determine the movement likely to take place in wall it is necessary to combine the individual effective movements due to thermal, moisture and other effects. However, the effective thermal and moisture movements are not directly additive since a wall is unlikely to be at both its maximum temperature and its saturated condition at the same time, so that to estimate the possible maximum movement it is necessary to consider carefully the temperature range over which the moisture movement occurs and make some attempt to combine the thermal and moisture movements on a rational basis rather than just considering the extremes. Since there are so many variables involved, it is extremely difficult to determine with any degree of certainty the actual movement that will occur."⁽¹³⁾

There is no proven mathematical method for determining movement. However, Copeland⁽¹⁶⁾ researched stress distribution in walls of different lengths and heights in an

attempt to determine the ratio of the effective maximum strain that is likely to occur in the wall, as a result of contraction, to the ultimate strain capacity of the wall. Copeland's paper is one of the few which tries to deal with the subject in mathematical terms and "may help to reinforce the engineering judgement upon which this subject must still heavily rely."⁽¹³⁾

In general it is simpler to adopt standard rules rather than to try to estimate movement.

Recommendations on the spacing of control joints to accommodate movement are given in Section 7.

6.0 Structural movement

6.1 Introduction

Dimensional changes (movements) apart from those associated with the aforementioned environmental factors, can be induced in masonry by one or more of the following:

- i) Directly applied loads.
- ii) Indirectly applied loads resulting from the deflection or shortening of interconnecting structural members or the creep and shrinkage of concrete elements.
- iii) Deformation and deflection of supporting elements such as reinforced concrete frames/beams and foundations.

Masonry is a brittle construction material with limited tensile strength. Although masonry relies on relatively large member sections to achieve adequate structural stiffness and stability, it is nevertheless a flexible material since its modulus of elasticity lies in a range between 0,5 and 6,5 GPa.⁽²⁴⁾ Masonry is however, not isotropic since it has different strengths and moduli of elasticity ⁽²⁵⁾ in two orthogonal directions.

In the design of structures, the magnitude of the abovementioned dimensional changes need to be carefully evaluated to ensure that cracking is minimised both in the masonry itself and in any element that it may support. In some instances, e.g. where a beam is supported at one end on a masonry column and at the other end on a concrete column, the assessment of the magnitude of movement before damage occurs is relatively simple.

However, when dimensional changes result from foundation movements, the assessment of allowable movement and associated damage can be extremely complex since factors such as the mode and magnitude of deformation, the geometry of the structure, the stiffness of the structure in shear and bending, the degree of tensile restraint and the engineering properties of masonry need to be evaluated.⁽²¹⁾

When designing both loadbearing masonry structures and structures incorporating masonry elements account must be taken of

- i) elastic deflections.
- ii) creep deflection.
- iii) long term shrinkage effects.
- iv) short and long term foundation movements.
- v) deformations arising from construction loads or construction sequences. (22)
- vi) thermal movements.

Furthermore, consideration should be given to the effect of structural movement on:

- i) visual appearance.
- ii) visible damage.
- iii) the function of the structure (i.e. the effects on impermeability, durability and insulation).
- iv) the stability of the structure.

6.2 Origins of cracks relating to structural movement

6.2.1 Foundations movements

Foundation movements on problem soils are normally associated with changes in moisture content (35):

- i) Expansive soils undergoes volume changes upon the wetting and drying of the soil horizons. The natural wetting up of the soil horizon below the structure may be sufficient to develop a mound profile underneath the structure. Alternatively, a change in moisture content due to the effects of climatic conditions and vegetation (evapo-transpiration) or a lowering of the water table may produce movements in soil horizons resulting in shrinkage movements.
- ii) Collapsible soils, upon saturation from an external source, will experience settlement. This settlement will take place rapidly if the soil is free-draining and gradually if it is not free-draining.

Uniform heave, shrinkage, collapse settlements or consolidation settlements generally do not cause damage to structures, but may detrimentally affect service (water and sewerage) pipe entries at the perimeter of structures. Non-uniform or differential movements may cause structural distress, deformations and overstressing of structural components resulting in the occurrence of damage to structures. (35)

Damage caused by heave/shrinkage movements differs from that due to collapse or consolidation settlements. Generally, if no precautions are taken in the construction of single storey residential

structures to reduce differential movements or to prevent conditions promoting potential movements from occurring, such movements will result in: ⁽³⁵⁾

- i) on expansive soils
 - damage occurring throughout the structure, the severity of the damage being greatest in the external walls, or internally in the central portions of the structure, depending on the moisture content of the soil preceding construction; and
 - cracks alternately opening and closing as a result of seasonal and climatic changes in the water content of the soil.
- ii) on compressible soils
 - damage manifesting itself in a particular portion of the structure, e.g. along a line through the structure; and
 - cracks opening in time as subsequent settlement occurs.
- iii) on collapsible soils
 - damage being confined to portions of the structure as and when collapse settlement occurs, e.g. beneath foundations adjacent to leaking pipes or adjacent to areas of poor drainage where ponding of rainfall occurs.

Observations in cases where mining subsidence occurred, indicate that minor damage, i.e. cracking of internal plaster and sticking of doors and windows, tends to occur when tensile and compressive ground strains exceed 0,04% and 0,08% respectively. ⁽²⁹⁾

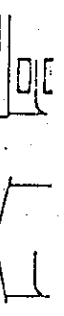
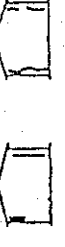
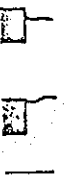
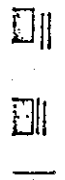
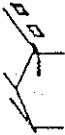
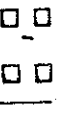
Less common forms of foundation movements relate to subsidence due to tunnelling, loss of support during underpinning, drag-down by adjacent structures, and movements around excavations. ⁽³⁴⁾

6.2.2. Cracking of loadbearing walls

Table 5 summarises the results of an extensive crack survey, together with the probable causes of the cracks, conducted in Sweden ⁽³²⁾ on some 150 two- to nine-storey apartment houses. Although the survey was confined to external walls constructed of light weight cellular concrete units laid in lime mortar, lime-cement mortar and without mortar (but with plastic shear connectors), most of the phenomena described could occur in any type of masonry construction to a greater or lesser degree.

It should be noted that more than 40% of the cracks in the survey relate to the interaction between concrete floors and masonry elements. Concrete

Table 5 : Survey of cracking in two-to nine-storey Swedish apartment houses (32)

Description	Type of crack	Probable cause of crack	N*(%)
1a Horizontal cracks at the topmost slab		Excessive shrinkage of the concrete slab. Rotation of the slab at the support.	12,1
1b Horizontal cracks at intermediate floor slabs		As for 1b. This crack occurs mainly where the vertical force is low eg under large windows.	9,9
2 Cracks in the foundation		Poor foundations	13,3
3 Vertical cracks at the corner of the building		When the concrete slab shrinks, the external walls are relatively free to follow the slab between the corners. However, at the corners, movement is prevented which results in high bending moments and tensile stresses.	10,2
4 Horizontal cracks around the windows of the uppermost story of building		Rotation of concrete slab at the support or effects of load eccentricity where the normal forces are low.	9,5
5 Cracks between roof and wall		Differential movement between the roof structure and the wall (thermal effects).	8,8
6a Cracks around staircases		Stress concentrations at section changes in walls	8,3
6b Cracks around balconies		As for 6a. Thermal movements.	5,0
7 Cracks in the mortar joints in the masonry		Differential movement in plaster layer and masonry itself	2,9
8a Vertical cracks below window openings		Movement in foundations. Difference between loads adjacent to and beneath windows. Drying effects of radiators beneath windows.	
8b Horizontal cracks in the portion of the wall below the window		As for 8a.	9,9
9 Horizontal cracks at the building corners		Vertical lift of the slab corners.	2,0
10 Vertical cracks at slab levels		Poor bedding of heavily loaded thin units. Units which have been disturbed during the placement of concrete slabs.	4,2

*N = Number of cracks of type
All crack observations

floors may cause cracking in supporting masonry elements due to one or more of the following:

- i) shrinkage of the slab itself.
- ii) rotation of the slab at the support.
- iii) eccentric transference of reactions of the slab into the walls.
- iv) vertical lift of the slab corners.
- v) thermal movements in the slab itself.

Most of the abovementioned are most pronounced where the vertical load in the masonry walling is low i.e. beneath windows and at roof level or where the spans of the floor are large.

6.2.3 Cracking of non-loadbearing walls

Non-loadbearing partition walls, provided that their ends are restrained, tend to distribute vertical loads to their supports in a similar fashion to an arch. BS 5977 : Part 1⁽⁴¹⁾ suggests that lintels over openings are only loaded by the loads and material contained in an idealised triangle with a base 1,1 times the clear span of the opening. Typically, such a triangle is considered to have slopes of 45 degrees.

Problems can arise, particularly in precast concrete floor slabs, where masonry partitions are constructed parallel to the span of the floor on top of it, since these walls tend to arch rather than follow the shape of the deflected slab as superimposed loads are applied.

An unsightly horizontal crack can develop where the bottom course or courses follow the deflection of the slab and separate from the remainder of the wall which arches over the deflected portion. In extreme cases, vertical cracking may occur as these courses tend to collapse.

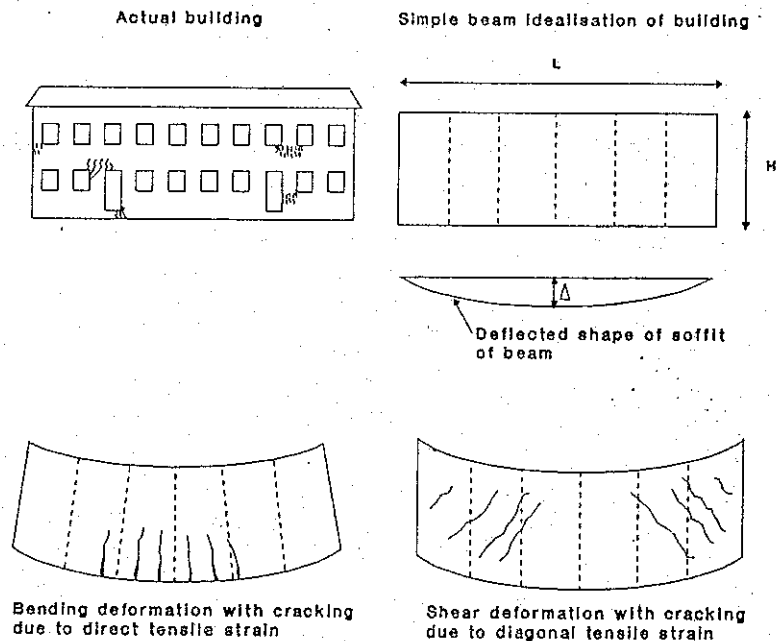
6.3 The onset of visible cracking in simple deep beams

Most damage due to structural movement manifests itself as cracking which results from tensile strain. The onset of visible cracking (as opposed to structural collapse) in a given material may be associated with a limiting, or 'critical', tensile strain (ϵ_{crit}). This critical tensile strain is not related to the strain at which loss of tensile strength occurs; nor does it necessarily represent a limit of serviceability. Nevertheless, it is useful to develop a simple model to predict the onset of visible cracking.

The application of the concept of limiting tensile strain can be illustrated by applying it to the cracking of a simple structure, such as a uniform, weightless, elastic beam of length (L), height (H) and unit thickness, which may

be thought of as representing a building (21,34) (See Figure 4). If the deflected shape of the beam is known, deflection criteria based on limiting tensile strains can be readily established.

Figure 4 : Cracking of a simple beam in bending and in shear



The simplest model is to consider the deflected shape of the beam soffit as being circular. The radius of curvature (R) may be related to the maximum deflection of the beam soffit (Δ) in accordance with the equation: $R = L^2/8\Delta$. If pure bending is occurring, then the bending strain $\epsilon_b = y/R$, where y is the distance from the neutral axis. When the neutral axis is in the middle of the beam, the maximum bending strain $\epsilon_{b(max)}$ occurs at $y = H/2$. Hence

$$\frac{\Delta}{L} = \epsilon_b \cdot \frac{4L}{H} \dots\dots\dots (1)$$

Two possible extreme modes of deformation are bending only, or shearing only (see Figure 4). Cracking associated with bending will occur as a result of direct tensile strain at the extreme fibre, whereas cracking associated with shear will result from diagonal tensile strain.

It can be shown that for a given deflection in a simply supported beam, the maximum tensile strains are not very sensitive to the precise form of loading. (21,34) The expression for the total deflection of a centrally loaded beam of unit thickness flexing in both shear and bending, is:

$$\frac{\Delta}{L} = \frac{P}{48} \cdot \frac{L^3}{EI} \left[1 + \frac{18}{L^2} \cdot \frac{1}{H} \cdot \frac{E}{G} \right] \dots\dots\dots (2)$$

where E is Young's modulus; G is the shear modulus; and I is the moment of inertia.

Equation (2) may be written in terms of the maximum extreme fibre strain $\epsilon_{b(max)}$ as follows:

$$\frac{\Delta}{L} = \epsilon_b \cdot \frac{L}{12y} \left[1 + \frac{18}{L^2} \cdot \frac{I}{H} \cdot \frac{E}{G} \right] \dots\dots\dots(3)$$

Similarly the maximum diagonal strain $\epsilon_{d(max)}$ can be written as:

$$\frac{\Delta}{L} = \epsilon_d \left[\frac{L^2}{18} \cdot \frac{H}{I} \cdot \frac{G}{E} + 1 \right] \dots\dots\dots(4)$$

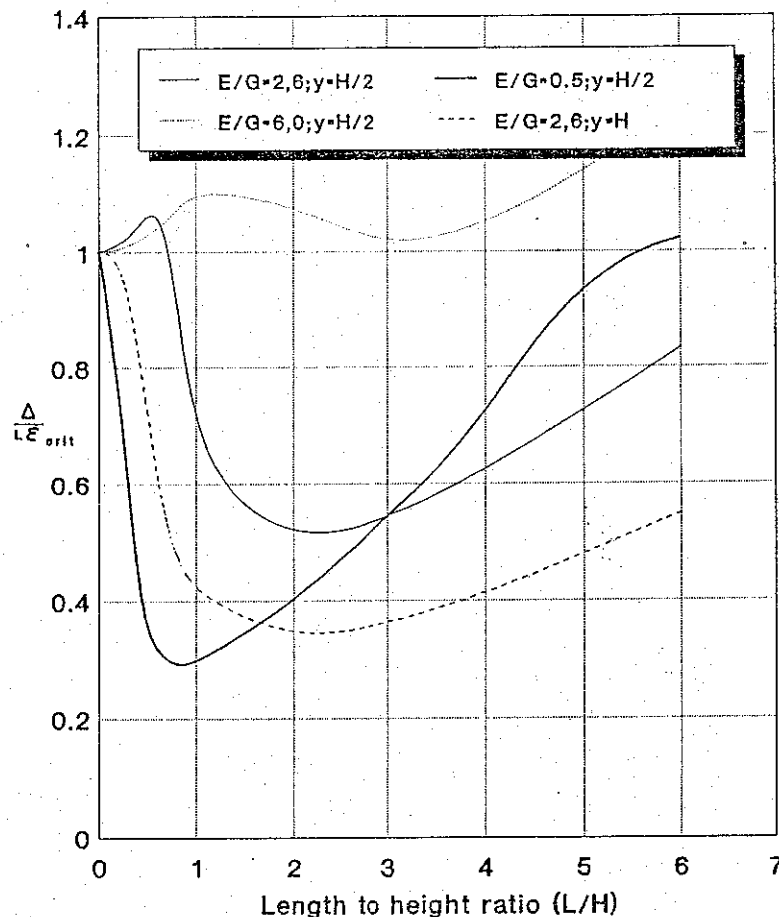
The limiting values of Δ/L for cracking of simple beams can be determined by setting $\epsilon_{(max)} = \epsilon_{(crit)}$ in equations (3) and (4). For any given value of $\epsilon_{(crit)}$, the limiting value of Δ/L (whichever is the lowest) depends on L/H , and the position of the neutral axis (y).

The ratio E/G equals $2(1+\nu)$ in simple isotropic theory, where ν is Poisson's ratio. E may be thought of as a measure of the longitudinal stiffness of the beam and G the stiffness of the beam in shear. ⁽³⁴⁾ Usually the ratio E/G is taken to be 2.6. However, in practice, the 'structure' may be built in such a way that the equivalent ratio E/G , in the beam model, does not conform to the simple isotropic elastic relationship. For example, a wall with a number of openings in it, will have relatively little shear stiffness, whereas a wall made of precast concrete units held together with dowels will be very stiff in shear ^(21, 34).

The location of the neutral axis has a pronounced effect on the value of the critical bending strain. Where walls are supported on other structural elements, such as beams, slabs and foundations, the neutral axis will tend to shift from the centre of the wall towards the base of the wall. Thus walls subjected to hogging moments will experience high tensile strains, whereas the same wall subjected to sagging moments will experience low tensile strains.

The effect of different values of E/G and the location of the neutral axis is illustrated in Figure 5. It can be seen that even for simple beams the limiting deflection ratio causing cracking can vary over wide limits, depending on the assumed stiffness of the structure and the location of the neutral axis. ^(31,34)

Figure 5 : Influence of E/G and the neutral axis on the cracking of a simple beam



6.4 Design recommendations

6.4.1 General

There is no single document that provides comprehensive design recommendations. Consequently, specialist literature and a range of codes of practice need to be consulted if a designer is to design a structure with a reasonable probability that the annoying phenomenon of cracking due to structural movement changes will not occur.

This section attempts to present a résumé of design recommendations that are given in codes of practice and specialist literature. These recommendations should not be viewed in isolation, but should be taken into account in arriving at the final design solution. For example, large distortions can be accommodated without unsightly cracking by appropriate detailing. However, such detailing will lead to a more flexible superstructure which will distort more on a given foundation. A superstructure not so detailed will usually be considerably stiffer and will thus distort less and, to that extent, be less at risk than might be otherwise expected. (29)

6.4.2 Structural codes of practice for masonry

The Australian masonry code (22), AS 3700-1988, clearly states the structural movements that should be controlled or isolated to avoid damage to masonry, the structure and its components, but offers no design recommendations. The British masonry code BS 5628 Part 1 (23) is completely silent on the subject whilst the South African code SABS 0164 Part 1, (5), only gives a value of 0,2 mm/m for horizontal shrinkage in lightly reinforced concrete slabs and 1,0 mm/m for combined shrinkage, elastic and creep shortening of reinforced columns and walls.

The American Building Code Requirements for Masonry Structures, ACI 530-88/ASCE 5-88, (26) states that 'masonry walls shall not be connected to structural frames unless the connections and walls are designed... to accommodate deformation'. The commentary on this code requirement suggests that clearance between the frame and masonry and flexible or slip-type connections may be used to accommodate movements arising from the elastic shortening of columns from axial loads, shrinkage or creep, the deflection of supporting beams, sideways in multi-storey buildings and foundation movements.

ACI 530-88/ASCE 5-88 requires that the deflection of lintels, due to dead and live load, should not exceed span/600, when providing vertical support to unreinforced masonry. The commentary to this document suggests that this requirement be waived with respect to reinforced masonry, since it is assumed that the crack width in the masonry will be controlled by the reinforcement. However, the American Uniform Building Code (20), in its chapter on Masonry requires that all elements that support masonry be designed so that their vertical deflection will not exceed span/600.

The British code of practice for reinforced masonry, BS 5628 Part 2 (24), states that 'fine cracks or opening up of joints may occur in reinforced masonry structures. However, cracking should not be such as to affect adversely the appearance or durability of the structure'. Apart from suggesting that movement joints be provided to accommodate the effects of temperature, creep, shrinkage and moisture movement, and thus minimise these fine cracks, the code suggests that deflections should not be excessive. A value for the final deflection inclusive of the effects of temperature, creep and shrinkage of span/125 for cantilevers and span/250 for other elements is recommended whilst a value of span/500 is suggested to accommodate the effects of deflection of the structure on partitions and finishes which occur after their construction.

(There seems to be wide acceptance that general deviations from the vertical or horizontal in excess of about $1/250$ are likely to be noticed ⁽²¹⁾. In vertical members constructional tolerances are generally of the same order ⁽⁵⁾ whereas local horizontal slopes exceeding $1/100$ are clearly visible ⁽²¹⁾. Thus the code recommendations of span/125 and span/250 relate more to visual appearance than to structural cracking).

6.4.3 Code of practice for reinforced concrete structures

BS 8110 in Part 1 ⁽²⁷⁾ provides basic span/effective depth ratios for beams, together with modification factors for reinforcement, for use in routine design.

The code states that 'these are based on limiting the total deflection to span/250 and this should normally ensure that the part of the deflection occurring after construction of finishes and partitions will be limited to span/350 or 20 mm ...'

Part 2 ⁽²⁸⁾ of the abovementioned code suggests that 'unless partitions, cladding and finishes, have been specifically detailed to allow for the anticipated deflections, some damage can be expected if the deflection after the installation of such finishes and partitions exceeds the following values:

- (a) $L/500$ or 20 mm, whichever is the lesser, for brittle materials;
 - (b) $L/350$ or 20 mm, whichever is the lesser, for non-brittle partitions or finishes;
- where L is the span, or, in the case of a cantilever, its length.

Furthermore, this code suggests that 'relative lateral deflection in any one storey under the characteristic wind load should not exceed $H/500$, where H is the storey height'.

6.4.4 Code of practice for the design of foundations

SABS 0161 ⁽³⁰⁾ requires that the designer ensures 'that the ratio of the maximum differential movement of a building, a foundation, or a foundation system to its length does not exceed the relevant value derived from Figure 2. Such calculated differential movement must not exceed 100 mm for any building, foundation, or foundation system unless special precautions have been taken to accommodate such movement'. (Figure 2 of SABS 0161 is reproduced as curves 1 and 5 in Figure 7).

6.4.5 Code of practice for general design procedures

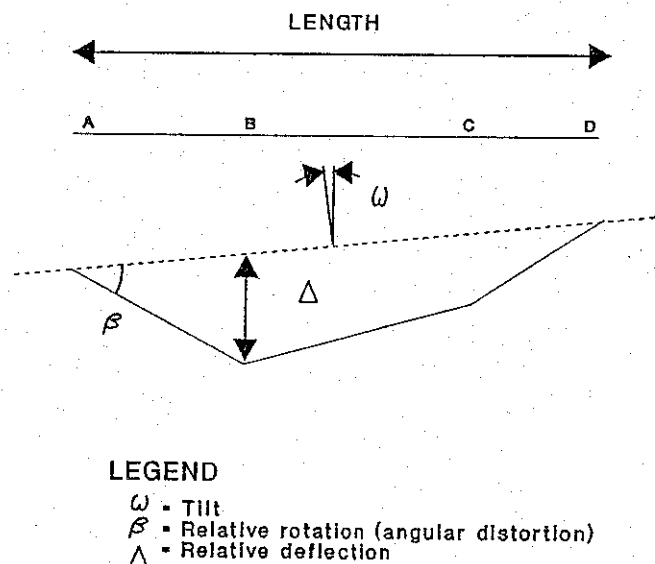
SABS 0160 ⁽³¹⁾ suggests the following deflection limits under expected service loads:

- a) Horizontal members
 Damage at supports (floors and roofs)
 : span/300
 Damage to partitions (floors and cantilevers):
 i) floor beneath partition : span/500 to span/300
 ii) floor above partition : 10 - 15 mm gap between partition and horizontal element.
- b) Vertical members
 Damage at supports : storey height/100.
 Damage to partitions : storey height/500.
 (A partition is defined as an 'internal vertical structure that is employed solely for the purpose of subdividing any storey of a building into sections, and that supports no load other than its own weight. SABS 0160 also limits deviations of horizontal floors, roofs and cantilevers and vertical members to span (or storey height)/250).

6.4.6 Specialist literature relating to foundation movements

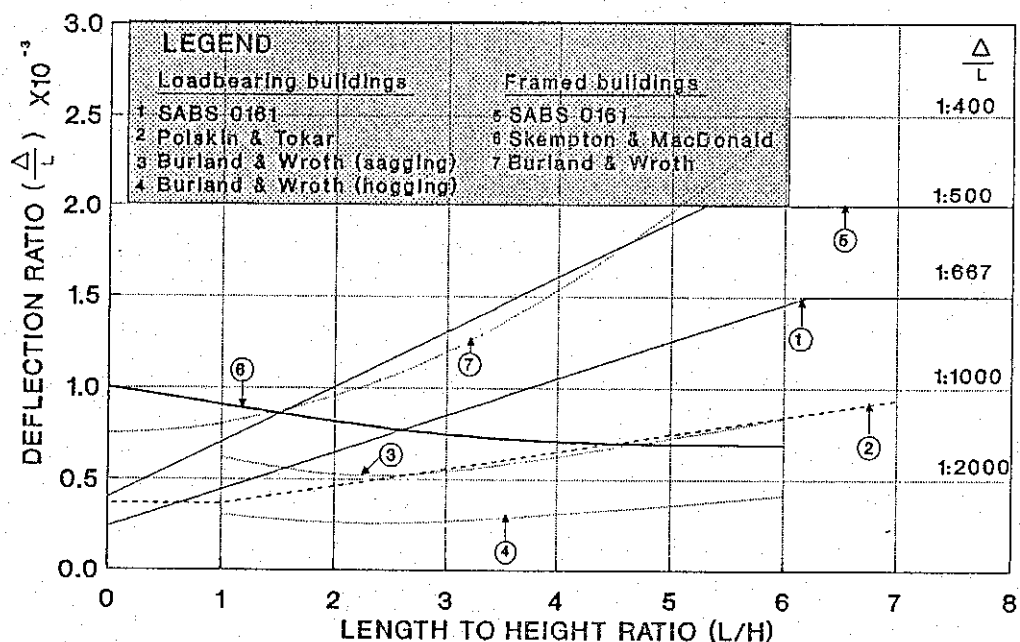
Various papers have been presented, during the last 30 years, on the subject of limiting deformations in buildings. These papers contain a wealth of information on case histories, as well as field data against which agreement between field observations and design recommendations has been obtained. Most of the data has centred around traditional steel and concrete reinforced frame buildings. Few measurements exist on the hogging deformation of load bearing masonry structures. Definitions of the typical terminology used are contained in Figure 6.

Figure 6: Terminology for allowable movements



Skempton and MacDonald, (43) on the basis of a survey of 98 buildings (mostly framed), of which 40 had been damaged due to settlements, recommend that to avoid cracking, angular distortion (β) should not exceed 1:500. Their findings indicated that an angular rotation of 1:300 would cause cracking in walls and partitions, whilst a rotation of 1:150 would cause structural damage. (Burland and Wroth (34) developed a relationship between angular distortion and the deflection ratio for simple beams. This relationship for an angular distortion of 1:500 is shown as curve 6 in Figure 7. It should be noted that in framed buildings, the finishes, including cladding will usually not be applied until some settlement has occurred. Consequently, in many cases, the limiting value of β could be reduced to account for the relative deflections that occur after the application of the finishes. This would be a function of the ratio of immediate to total settlement (21).

Figure 7 : Relationship between Δ/L and L/H for buildings subjected to foundation movements



Polskin and Tokar (42), on the basis of a limiting tensile strain of 0,05%, recommend that curve 2, in Figure 7, be used for the design of loadbearing walls.

Burland and Wroth, (21,34) on the basis of an analysis of a simple beam (equations (3) and (4) in Section 6.3) and a limiting tensile strain of 0,075% produced curves 3, 4 and 7 in Figure 7 and compared them against previously published design criteria

and field data for framed buildings and loadbearing buildings experiencing sagging and hogging deformations respectively. They also carried out a survey of data relating to the cracking of infill frames and masonry walls and concluded that the range of values of average tensile strain at the onset of visible cracking, for a variety of common building materials, was remarkably small. For brickwork and blockwork set in cement mortar ϵ_{crit} lies between 0,05 and 0,1%, while in the case of reinforced concrete the values lie between 0,03 and 0,05%, notwithstanding wide variation in strength.

They confirmed the recommendations made by Skempton and MacDonald, (which may be conservative for $L/H > 3$) for framed buildings, and those made by Polskin and Tokar for loadbearing masonry subjected to sagging, but recommend that curve 4, in Figure 7, be used for loadbearing structures subject to hogging. (Burland and Wroth (21,34) assumed: E/G was equal to 2,6 in all cases; y was equal to $H/2$ for sagging deformation modes and equal to H for both framed buildings and hogging modes of deformation; tensile restraint in framed buildings).

6.4.7 Specialist literature unrelated to foundation movements

6.4.7.1 Concrete framed structures

In X/Bou 2-53 (35) it is suggested that the total shortening of a reinforced concrete frame after completion of masonry cladding, in most adverse practical conditions, will not exceed 0,12% i.e. 1,2 mm/m. A design value for the maximum residual shortening of 1,0 mm/m is suggested where concrete aggregates do not have excessive shrinkage characteristics. (38)

6.4.7.2 Non-loadbearing walls

Schild et al (33) make the following recommendations with respect to non-loadbearing partition walls:

- i) Floor slab thickness in metres should not be less than the square of the span in metres divided by 150.
- ii) The construction of walls should be delayed as long as possible after the formwork to the floor slabs has been struck.
- iii) Unnecessarily large spans between supporting structures (greater than 7,0 m) should be avoided.
- iv) Movement joints should be provided at the top of walls beneath floor and roof slabs.

Curtin et al, (39) suggest that consideration should be given to the provision of bed joint reinforcement in the lower bed joints and a slip membrane between the wall and the floor as well as between the wall and the screed. This would allow, particularly in the case of block partition walls, arching to take place without any separation of the bottom course.

They also recommend that the bond between cladding (particularly, burnt clay) and supporting concrete elements, be broken to prevent spalling of the concrete elements, particularly at the corners of structures.

6.4.7.3 Loadbearing walls

Most authorities (32,33,39,40) recommend that long spans be avoided and slip layers be provided beneath roof slabs.

6.5 Design for cracked structures - an alternative solution to foundation movement

Jennings and Kerrich (44) stated the following in regard to the economic consequences of damage in buildings founded on heaving soils. "Attention is drawn to the curious perfectionist outlook that is common in all areas where cracking of buildings is reputedly serious. In such localities one encounters the attitude that to be satisfactory a building must be completely free of cracks, even of the most insignificant type. In other areas, where cracked buildings are not a major problem, the same observers are prepared to accept much more unsightly cracks without deeming them to be particularly damaging to the building".

Pryke (29) observed in South East England that his company was rarely consulted about cracks (relating to differential settlement) "unless at least some had reached about 3 mm in width. Often a householder will not notice a crack until his attention is drawn to it by a surveyor employed in connection with a proposed sale".

Clearly, within acceptable limits, there is economic justification for tolerating cracks in structures. Engineers, developers, and owners ought to ask the question: is total prevention of cracks cheaper than repair?. Watermeyer and Tromp (35) have proposed a classification system for damage to residential masonry structures founded on problem soils (see Table 6). This classification system categorises degrees of expected damage in specific terms of ease of repair and structural distress, and can serve as a standardised specification of structural performance governing the engineering design. At the same time, it can be used to determine the expectations of the owner and/or developer.

Table 6 : Classification of damage to residential structures founded on problem soils

Description of damage in terms of ease of repair and typical effects	Approximate maximum crack width in walls (mm)	Category and degree of expected damage
Minor damage - Categories 0 to 2		
Hairline cracks less than about 0,1 mm width are classed as negligible.	< 0,1	0 Negligible
Fine internal cracks which can easily be treated during normal decoration. Cracks rarely visible in external masonry.	< 1	1 Very slight
Internal cracks easily filled. Redecoration probably required. Recurrent cracks can be masked by suitable linings. Cracks not necessarily visible externally. Doors and windows may stick slightly.	< 5	2 Slight
Significant damage - Categories 3 to 5		
Cracks can be repaired and possibly a small amount of masonry may have to be replaced. Articulation joints may have to be cut in some of the walls. Doors and windows sticking. Rigid service pipes may fracture. Weather-tightness often impaired. Up to 10 mm gap between ceiling cornices and walls.	5 to 15 (or a number of cracks 3 to 5 in one group)	3 Moderate
Extensive repair work which includes breaking out and replacing sections of walls, especially over doors and windows, cutting of articulation joints in walls, and the construction of moisture trenches and apron slabs around the structure, or the jacking of foundations, depending on the type of soil movement. Window and door frames distorted, floor sloping noticeably. Walls leaning or bulging noticeably, some loss of bearing in beams. Service pipes probably disrupted. Up to 20 mm gap between ceiling cornices and walls.	15 to 25 (depending also on number of cracks in a group)	4 Severe
Major repair work required, involving partial rebuilding and the abovementioned repair techniques. Beams loose bearing, walls tilt badly and require shoring. Windows broken and distorted. Danger of instability.	Usually greater than 25 (depending also on number of cracks in a group)	5 Very severe

NOTE:

Crack width is only one factor in assessing damage and should not be used on its own, as a direct measure of damage. In assessing the degree or severity of damage, account must be taken of the location in the structure where it occurs, and also of the function of the structure.

The recommendations given in section 6.4.6 are aimed at achieving a crack-free structure. Generally, observance of the deflection criteria presented in Figure 7 will result in structures experiencing no cracking or isolated damage more severe than Category 0 (negligible), as described in Table. The Australian standard for residential slabs and footings, AS 2870 ⁽³⁷⁾, in an appendix states that 'it is impossible to design a footing system that will protect the house from movement under all circumstances' and suggests that footing systems designed in accordance with the standard should in most situations experience damage no more severe than Category 1 (Very slight).

In South Africa, design recommendations for the construction of residential structures on heaving clays are based on criteria developed for crack-free structures. Table 7 summarises typical allowable deflection ratios used in South African ⁽³⁶⁾ and Australian practice. ⁽³⁷⁾ A shift in expected damage from category 0 to category 1, in the case of solid masonry, would result in a foundation structure with 43% less stiffness. (Interestingly, a Cement and Concrete Association of Australia publication ⁽⁴⁵⁾, recommends that, in the case of non-articulated masonry, the deflection ratio of 1:2000 can be reduced to 1:1500 where walls are fair-faced and constructed in cement-lime mortar).

Table 7 : Residential structures on heaving clays - Allowable deflection ratios

TYPE OF STRUCTURE	Allowable defection ratio Δ/L	
	Williams et al ³⁶	AS 2870 ³⁷
Clad frame	-	1:300
Articulated masonry veneer	1:750	1:400
Masonry veneer	-	1:600
Articulated full masonry	1:1000	1:800
Full masonry	1:3500	1:2000

As indicated above, the onset of visible cracking does not necessarily represent a limit of serviceability. Provided the width of cracking is controlled by some form of tensile restraint, (eg reinforcement, concrete frame or foundation), it is possible that significantly larger deformations of masonry can be tolerated than those giving rise to initial cracking. Reinforced concrete beams and slabs are good examples where limited cracking is regarded as normal, and guidelines are laid down for the maximum acceptable width of crack in various circumstances.

Uncontrolled cracking due to hogging of walls should be prevented. Once a crack forms at the top of the wall, there is nothing to stop it propogating downwards. It is

therefore advisable to provide bed-joint reinforcement within the joint immediately above windows and openings and also in the uppermost joint in a continuous band around the structure, both in the internal as well as in the external walls. BS 5628 : Part 2 ⁽²⁴⁾ recommends that the maximum size of bar in bed joints should not exceed 6 mm. In practice, 5,6 mm prestraightened hard drawn wire (obtainable from a welded steel fabric reinforcement manufacturer) is well suited for South African structures and will offer considerable restraint.

7.0 Accommodation of movement by use of control joints

7.1 General

One of the best ways of ensuring that the masonry is able to accommodate small seasonal movements due to temperature and moisture changes is to design the building so that the masonry is separated into discrete panels by the provision of control (movement) joints i.e. to reduce stresses by reducing restraint.

Control joints may be orientated horizontally or vertically and may be further classified as expansion or contraction joints. Alternatively, a control joint may be provided in the form of a slip joint separating dissimilar materials to prevent excessive shear forces being generated between the dissimilar materials.

Control joints should be designed so that movement can take place without transferring stresses across the joint. Their design and location must be such that the structural and functional (i.e. impermeability, sound insulation and fire resistance) integrity of the walling is not impaired. Caulks and sealants to these joints should both be able to adequately seal the joint against moisture penetration and accommodate any deformations to which the joint may be subjected. (A caulk refers to a material suitable for filling compression joints, whereas a sealant refers to a material suitable for sealing joints which may experience revisable strains).

Where necessary, dowels, angles or channels strong enough to provide lateral stability should be incorporated. The dowels, which are usually metal rods or flat strips, should be anchored into the masonry in such a way that longitudinal movement is not restrained. Angles or channels fixed on to one side of the control joint should project into grooves and recesses so as not to restrict longitudinal movement.

Vertical control joints are generally provided at regular intervals in long lengths of walls whereas horizontal joints are usually provided beneath horizontal concrete members in framed structures.

7.2 Location of vertical control joints

These joints in unreinforced masonry should generally be positioned where concentrations or changes in stress occur, i.e. at: openings; major changes in wall height; changes in wall thickness; control joints in foundations, floors and roofs; near wall intersections and near return angles in L, T and U-shaped structures.

In reinforced masonry, vertical control joints should be considered at: changes in wall thickness; major changes in wall height; at control joints in foundations, in floors, and in roofs; and at wall openings.⁽¹⁹⁾

Generally, vertical control joints are not located at the extreme corner of wall returns for stability reasons. They do not usually continue below ground floor damp proof courses where changes in temperature and moisture content are minimal and are not normally required in interior walls of dwellings⁽¹⁴⁾ where the effects of thermal expansion may be disregarded.⁽¹⁸⁾

Control joints should be built into the wall during construction and run the full height of the masonry. Sawn joints are generally more expensive, require great care in cutting and are not normally as effective as built-in joints.

The position of control joints, bond beams and joint reinforcement should be clearly shown on the plans.⁽¹⁹⁾

7.3 Location of horizontal control joints

Horizontal control joints are normally provided at every storey height in reinforced concrete framed buildings where the cladding comprises burnt clay units or where the shortening of columns can result in excessive loads being transferred from structural members onto masonry walls.

Horizontal control joints may also be provided to separate dissimilar materials.

7.4 Design recommendations

7.4.1 General

Table 8 summarises potential movements due to environmental and material factors that individual units may experience.

Table 8 : Summary of potential movements of masonry units

Unit type	Movement type	Potential Movement (mm/m wall length)
Burnt clay	Thermal Moisture expansion	0,12 to 0,24 ** 0 to 0,5 (Category 1) 0,5 to 1,0 (Category 2) 1,0 to 2,0 (Category 3)
	Moisture movement	0,1 to 0,2*
Calcium Silicate	Thermal Drying shrinkage	0,33 to 0,45** 0,45
	Moisture movement	0,12 to 0,18*
Concrete	Thermal Drying Shrinkage	0,21 to 0,42** 0,60
	Moisture Movement Carbonation (shrinkage)	0,12 to 0,18* 0,1 to 0,2

* Reversible movements

+ 30°C temperature change

As a general rule, 10 to 12 mm wide vertical joints to accommodate these horizontal movement should be provided in unreinforced walls at the intervals given in Table 9. (5, 14) However, there are wide differences in the physical properties of concrete units and other joint spacings may be acceptable e.g. SABS 0145⁽⁶⁾ permits joint spacings up to 9m for unreinforced walls "subject to the length of wall between control joints not exceeding twice the height of the wall". Vertical load on walls will tend to increase the spacing of these joints.

Table 9 : Spacing of vertical control joints in horizontal lengths of unreinforced wall

Unit type	Moisture expansion	Approximate spacing of 10-12mm joints, m	
		Walling	Parapets
Burnt clay	<0,05%	16,0	11,0
	0,05% - 0,10%	10,0	8,0
	0,10% - 0,20%	6,0	4,0
Calcium silicate	-	7,5 - 9,0	6,0
Concrete	-	5,0 - 7,0	5,0

SABS 0164 recommends that the distance from the corner of return in the outer leaf of a cavity wall to the first vertical joint should not be more than half the values given in Table 9.

The ACI commentary on building code requirements for concrete masonry structures recommends that control joints in concrete masonry to accommodate expansion be provided at a spacing of between 45m to 60m.

Bed joint reinforcement in the form of brick force or steel rods in each skin (2,85 - 6mm diameter) can be used in some instances to modify the joint spacings. Horizontal joint reinforcement does not eliminate cracking, but redistributes the stresses when cracking commences, resulting in a fine evenly distributed, barely visible cracks.

BS 5628 Part 3 suggests that where bed joints reinforcement is used in burnt clay masonry, joint spacings in "excess of 15 m are satisfactory but expert advice should be sought". SABS 0145⁽¹⁶⁾ offers some quantitative guidance on the spacing of reinforcement required to increase control joint intervals in concrete masonry walls without openings. In terms of SABS 0145, joint intervals can be increased to between 13,5 and 18,5 m if horizontal reinforcement is provided between 600 and 200 mm vertical centres.

7.4.2 Joints in framed buildings

The code of practice for the application of the National Building Regulations, SABS 0400⁽¹⁾ requires that movement joints, at intervals of not more than 10 m, be provided in external masonry cladding and infill panels in framed buildings to allow for relative horizontal movement. Furthermore, this code requires that such cladding be supported on suitable beams, slabs or nibs at each storey and that "adequate provision be made for relative vertical movement between the masonry and the structure frame at the underside of such supports".⁽¹⁾

SABS 0164 Part 1 ⁽⁵⁾, on the other hand, recommends that vertical control joints be provided in accordance with Table 9, but suggests that in the case of clay masonry, an assessment based on available masonry-unit expansion data, the modulus of elasticity of masonry and restraint conditions, be carried out to determine whether all of the joints are required.

In terms of horizontal joints, SABS 0164 states that:

- i) These joints are not normally provided for internal walls but warns that the shortening of columns and the expansion of masonry can result in the transference of load from the concrete frame to the masonry.
- ii) The need for these joints in walls constructed of calcium silicate or concrete units should be checked as the shortening of the framework may exceed the shrinkage of the masonry.
- iii) For external solid walls or the internal leaf of cavity walls constructed of clay units, these joints should be provided under every horizontal member if an assessment shows that excessively high compressive stresses can develop. Nevertheless, joints should be provided at every 4th floor or 12m whichever is the lesser.

BS 5628 Part 2 (24) gives a relationship between the characteristic compressive strength and the modulus of elasticity of masonry. If slenderness ratios and eccentricities are ignored and an average value of 3,2 is adopted for the partial factor of safety for materials, it can be shown that the following movements can result in compressive stresses being induced in walls which exceed the permissible compressive strength of masonry:

- short term (all units) 0,35 mm/m
- long term (calcium silicate units) 1,0 mm/m
- long term (clay units) 0,7 mm/m

Comparing the abovementioned limits to potential movements that masonry units may experience (see Table 8) and design values for residual frame shortening (see Section 6.4.7), it would appear that it is somewhat difficult to justify the omission of horizontal joints at every storey height in clay masonry unless the partial factor of safety for materials is reduced.

7.4.3 External cavity walls in framed structures

BS 5628 Part 1 suggests that "the uninterrupted height and length of the outer leaf of external cavity walls should be limited so as to avoid undue loosening of ties due to differential movements between the two leaves. The outer leaf should, therefore, be supported at

intervals of not more than every third storey or every 9m, whichever is less. However, for buildings not exceeding four storeys or 12m in height, whichever is the lesser, the outer leaf may be uninterrupted for its full height." A horizontal control joint must be provided under every horizontal member that gives support to the outer leaf of the wall.

8.0 Conclusion

For masonry to perform satisfactorily excessive build-ups of stress and unanticipated cracking must be avoided. At the design stage possible movements and stress concentrations in walls should be predicted and allowance be made to keep these within acceptable limits. The use of control joints for this purpose is good practice.

The success of control joints depends on their spacing, their configuration, the use of correct infilling and sealing materials, while workmanship throughout must be of adequate standard.

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APPENDIX A : Standard Masonry specifications

SABS	227 (1986)	285 (1979)	1215 (1984)	COMMENT
PRODUCT PROPERTY	BURNT CLAY MASONRY UNITS	CALCIUM SILICATE MASONRY UNITS	CONCRETE MASONRY UNITS	
DIMENSIONS	L W H 190 x 90 x 65 +190 x 90 x 90 288 x 90 x 65 +288 x 90 x 90 288 x 90 x 139 288 x 139 x 90 288 x 139 x 139 222 x 106 x 73 222 x 90 x 114 222 x 140 x 73	L W H 222 x 106 x 73 190 x 90 x 90 290 x 90 x 90 290 x 190 x 90	L W H *190 x 90 x 90 *290 x 90 x 90 *390 x 90 x 190 *390 x 190 x 190	+Preferred modular size *Recommended nominal dimensions
TOLERANCES (mm)	± 7 ± 4 ± 4 ± 5 ± 3 ± 3	± 2	- 2 ± 3 ± 3 - 4	FBS - SABS 227 FBX - SABS 227
NOMINAL COMPRESSIVE STRENGTHS (Mpa)	7.0 10.5 14.0 17.0 (Engineering units) 7.0-49.0	7 14 21 35	3.5 7 10.5 14 21	Refer National Regulations on use of difference strength units
GRADES OF EFFLORESCENCE	No spec*	No spec	No spec	*Test for soluble salts to determine efflorescence: Nil, slight, moderate, heavy. Explanatory note in appendix.
DRYING SHRINKAGE	no spec	.045% Max	.06% Max	
MOISTURE EXPANSION	No spec*	No spec	On rewetting 0.02%	*Moisture expansion test detailed
WATER ABSORPTION	No spec*	No spec	No spec	*Water absorption tests specified
SOUNDNESS	Surface pop-outs specified	No spec	Surface pop-outs specified*	*Test specified where units contain slag or clinker or burnt clay brick aggregates

Notes

1) Where there is no specification, it is customary for the purchaser/supplier to agree on a standard.

2) Burnt clay masonry units

2.1) Classification

FBS - Face Brick standard

These are clay face bricks that are durable, uniform in size and shape and require no further decorative or protective treatment.

FBX (Extra) - Face Brick Extra

Durable face brick possessing the highest degree of size, shape and colour uniformity.

FBA - Face Brick Aesthetic

These are durable clay face brick selected or produced for a highly individual aesthetic look derived from deliberate non-uniformity of shape and colour.

NFP - Non-Facing Plastered

Clay bricks that are suitable for general building work which is then to be plastered or rendered.

NFX (Extra) - Non-Facing Extra

These bricks are for use in general building work where clay bricks are required mainly for durability such as in areas below ground and damp-proof course level and in damp situations such as retaining walls and where looks are relatively unimportant.

ENGINEERING/MPa

Usually solid bricks. Available at various strengths from 7 to 49 MPa for non-aesthetic structural use.

2.2) Shape, Colour, Texture options

Special shapes to match certain products are available for use in architectural detailing. South Africa is fortunate in having a wide range of coloured and textured face bricks comparable to anywhere in the world.

3) Calcium silicate masonry units

Calcium silicate masonry units are classified according to:

- Compressive strength (MPa)
- Dimension (work size)
- Colour (as agreed between manufacturer and purchaser)
- Texture (smooth or rock face)

Calcium silicate masonry units are manufactured in factories in Cape Town and Durban.

4) Concrete masonry units

Concrete masonry units are classified according to:

- Compressive strength (MPa)
- Shape (solid or hollow)
- Dimensions (work size)
- Colour (as agreed between manufacturer and purchaser)
- Texture (smooth or rock face)
- Profile (plain, scored, fluted, ribbed)

The range of masonry units available will vary considerably from one manufacturer to another, depending on local needs and building practice.

Specify that units comply with SABS 1215 Concrete masonry units and preferably manufacturer holds the SABS mark for his products.