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ANALYSIS OF LOW-RISE MASONRY WALLS
SUBJECTED TO THERMAL AND MOISTURE DEFORMATIONS

by

© Mohamed Abdalla MOHAMED, B.Sc.

A thesis submitted to the
Faculty of Graduate Studies and Research
in partial fulfillment of the requirements
for the degree of
Master of Engineering

FACULTY OF ENGINEERING
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March 1984

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SUBJECTED TO THERMAL AND MOISTURE DEFORMATIONS"**

submitted by Mohamed Abdalla MOHAMED, B.Sc.

in partial fulfilment of the requirements for

the degree of

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Abstract

Deformations of masonry materials, elements and structures may be due to the following:

- (a) Temperature change resulting in expansion or contraction.
- (b) Moisture content changes resulting in swelling or shrinkage.
- (c) Applied loads resulting in elastic and inelastic deformations.
- (d) Chemical action in the presence of moist air or water resulting in volume change.

Although it is difficult to determine the exact deformations or stresses created by the above factors, it is possible to compute the temperature and moisture effects approximately, if factors (c) and (d) are ignored.

The first part of this thesis presents a summary of the literature dealing with the deformations caused by the above mentioned factors. In the second part, thermal and moisture effects on masonry walls are numerically analyzed as an equivalent change in the wall temperature.

To investigate the deformations of low-rise masonry walls, a two-dimensional finite element analysis was carried out. Seven cases were assumed to represent different boundary conditions such as, different aspect ratios, presence of openings, and effects due to top restraint, reinforcement, temperature profile and load-bearing situations.

In the analysis, walls were subjected to a linear temperature profile and masonry was assumed to be an elastic material. The walls considered had height to width ratios ranging from 0.1 to 0.8 representing a wide range of practical situations.

The results indicate that higher stresses and potential cracking were most affected by the degree of restraint, by the presence of openings, by the assumed temperature profile, and by the aspect ratio.

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CHAPTER I

INTRODUCTION

L1 GENERAL

A knowledge of deformations that can take place in masonry is essential to an understanding of its structural behaviour. Furthermore, as masonry frequently supports, or is supported by, or is tied to other structural materials, an estimate of the relative deformations that may occur is essential to an evaluation of building performance.

Movement in buildings is becoming of prime concern to the construction industry because of the serious consequences it may have on safety, serviceability and maintenance costs. The growth in importance of movement can be attributed to the new forms of construction including prefabrication and the trend towards taller and more slender buildings.

In spite of its importance the subject has received remarkably little attention in the past and even today it is difficult to find a book dealing specifically with the different aspects of movement in buildings.

The basic causes of movements in buildings are the natural laws which govern the behavior of materials. Contrary to the popular belief that buildings are inert, they do in fact respond quite significantly to changes in loading and environment. To the engineer, these movements should be an important consideration in ensuring that the structure is safe and will satisfactorily fulfill its purpose during its service life.

In important structures special care is taken to allow for such movements, but in general building construction involving masonry, the problem is rarely considered in the depth it should, and often it is overlooked or ignored completely until serious problems have developed.

Older massive methods of masonry construction generally involved less movement and the forms of construction were less susceptible to damage arising from such movement. Modern lighter building techniques, however, are in many important respects radically different from those of the past, and generally lead to greater movements in the structure.

Changes in temperature and moisture content produce movements which tend to occur towards, or away from, the stiff portions of the building which act as fixed points. For long low-rise masonry structures, of special importance is the differential movement between masonry walls and the foundation as well as between masonry walls and the roof.

1.2 PURPOSE AND SCOPE

In attempting to make allowance for movements in buildings the designer is faced with certain problems. In theoretical analysis work, the designer deals with each movement factor separately, whereas in practice the movement factors may interact and it may be difficult to assess to what degree they interact. Secondly, it is often difficult to predict movements in a complex building on the basis of controlled laboratory tests on a number of elements.

To achieve a reasonable degree of accuracy, stress predictions should be based on the analysis of the whole building as an entity. For this reason measurements made on buildings under construction and during their subsequent life are of utmost importance. Such field measurements have been carried out only in a few isolated cases; generally it is too difficult and expensive to measure movements in whole structures or in major components once the building is in service. Added to this is the fact that much basic information is still lacking, in particular as regards to moisture movement, creep and temperature gradients.

In spite of this it is believed that numerical analysis work can give a very useful insight into the problem of movements in masonry walls and in some cases may even provide a reasonably accurate assessment. Furthermore, a thorough understanding of the various factors and principles involved will assist the designer to take measures in his design and constructional detail which will minimize the harmful effects of movements in buildings.

The problem of thermal and moisture stresses in low-rise masonry walls is the subject of this thesis. The work is presented in the following sequence:

- i. Literature review covering a wide number of publications describing the state of knowledge over the past several decades.
- ii. A survey of finite element analysis work relating to masonry movements.
- iii. Assembly of a FORTRAN program capable of solving internal in-plane strains that are not a result of applied loads.
- iv. A numerical finite element analysis of typical wall cases including different aspect ratios, openings, restraint conditions and thermal profiles.
- v. Conclusions based partly on past experience and partly on the finite element analysis work dealt with in this thesis.

CHAPTER II

LITERATURE REVIEW

II.1 GENERAL

The various elements and materials that make up a building are in a constant state of motion. All building materials expand and contract due to changes in temperature; some materials move with change in moisture content; unrestrained materials change dimensions due to stress and some tend to flow when subjected to sustained loads.

A building is a dynamic structure, the successful performance of which depends upon design as well as construction that allows movement of elements and the whole without exceeding the permissible stresses.

The general problem in designing a building is to allow for potential movements such as thermal movements, foundation settlement, elastic and creep movements under load, chemical expansions and frost effects.

The movements must be considered both in the masonry material, block and brick units, as well as in the masonry assemblage.

It is a common fallacy to think of materials as good or bad, durable or non-durable, strong or weak as if these were inherent properties of the materials. It is seldom that simple or definite. These terms are relative; durability is related to the conditions of exposure. The service life for any particular usage will depend on the severity of the conditions to which the material is exposed. No material is infinitely strong; all will crack and fail under some definite level of applied load. If we do not want materials to crack, we must assure that they are not subjected to that level of loading.

It is the purpose of this chapter to discuss the various factors that cause movements in low-rise masonry walls and to summarize the previous work reported in the literature.

II.2 MOISTURE MOVEMENTS

Materials capable of adsorbing water expand as they do so and contract again on drying. As many common building materials have a porous structure and can adsorb water more or less readily, the nature and magnitude of moisture deformations assume considerable importance. Moisture deformation, like thermal deformation, is generally reversible, except in such materials as concretes, mortars and plasters. For these the initial shrinkage that occurs during drying in curing may be considerably greater than any following reversible deformation.

Moisture movement is one of the factors that affect the serviceability and the structural performance of masonry and, acting individually or together with other factors, they can result in stresses which may exceed the permissible stresses. Where permissible stresses are exceeded, cracking may take place which in turn may lead to serious serviceability and possibly also safety problems.

II.2.1 MOISTURE MOVEMENTS IN CLAY BRICK

Clay brick responds to moisture changes by volumetric change. These changes are connected with the manufacturing process and greatly affect the quality of the bricks.

During manufacturing the plastic clay brick shrinks while being predried before entering the kiln and also during firing. While cooling in the kiln, it adsorbs moisture and expands.

MOORE and NOBLE, [1], VAUGHAN and DINSDALE, [2], have argued that this expansion begins at high temperature. Before firing, the plastic clay brick may be conceived as consisting of small particles of different sizes surrounded and separated by water films.

In the early stages of drying, water is evaporated from the surface of the clay and is drawn from the interior by surface forces. During this period the thickness of the water film decreases, the particles are drawn closer together, and if the sample is dried slowly enough, the decrease in volume is equal to the volume of water removed. Little further contraction can take place if the thickness of the film is reduced to a minimum. The water that is subsequently removed comes mainly from pore spaces or from interstices between the particles until drying is complete.

Because of uneven drying, the outside of the brick will shrink more than the inside in the initial stages leading to tension in the outer shell and compression in the inner zone. If these tensile stresses exceed the green strength of the clay, cracking will occur. Experiments were reported by STEVANS and HOLLAND, [3] in which they obtained more direct evidence by measuring with silica rods and comparing length changes of two specimens as they cooled from 800° C in the kiln, one cooled in the air and the other in a vacuum at about 400° C. The first showed less thermal shrinkage than the second one which indicated that moisture expansion had commenced.

In practice, however, moisture may also initiate chemical reactions which deposit salts resulting in volume expansion of the brick. Frost action as well may result in volume change.

II.2.1.1 REVERSIBLE WETTING AND DRYING EFFECTS

The reversible dimensional changes in a brick due to wetting and drying are a result of the interaction between water molecules and the internal surfaces of the material. For brickwork the magnitude of this movement between drying and saturated conditions is typically in the range of 0.007% to 0.02% according to BESSEY, [4]. The greater the concentration of water molecules in the atmosphere, the greater the amount of water adsorbed by the surface; and the larger the internal surface area of the material, the more absorptive it will be. The gradient of moisture content is the cause of the differential movement which in turn may induce warping or stresses. The concentration of water vapour in the atmosphere is expressed by the relative humidity.

It has been established by BESSEY, [4] and HOSKING et al, [5], that moisture expansion is primarily due to chemical reactions between water and certain constituents of the ceramic bodies, the most significant being morphous aluminosilicates, amorphous silica, γ -alumina and glass.

BESSEY, [4], reported that some of these reactions are not reversible at normal temperatures, thus accounting for the permanent nature of expansion. Some are far more active than water vapour thus accounting for the greater expansions obtained when autoclaved than when exposed to natural atmospheric conditions.

II.2.1.2 FACTORS AFFECTING PERMANENT MOISTURE EXPANSION

i. Time of Exposure to Moisture

The magnitude of moisture expansion increases with the time of exposure to moisture; there is an initial rapid period of growth which reduces with time.

HOSKING et al, [5], reported a period of expansion followed by a reduction in length but a subsequent paper, [6], explained that this erroneous conclusion was formed because of wear in measuring instruments.

ii. Time of Laying

Walls built with bricks that have been allowed to stand for some period of time after leaving the kiln show less expansion than walls built with bricks fresh from the kiln. BEARD et al, [7] found that walls built with kiln fresh brick had expanded after six years to about 1.2 times that of walls built with 14 days old bricks.

After one year, walls built with fresh bricks by LAIRD and WICKENS [8] had expanded about 2.5 times that of one month old bricks. Similar results are reported by SMITH, [9].

iii. Moisture Temperature

Increasing the temperature of the moisture to which the brick is exposed increases the rate of permanent moisture expansion. FREEMAN and SMITH, [10] observed that the expansions of bricks after 238 days of exposure to the atmosphere at 18° C were approximately the same as expansions obtained in 6 hours of exposure to steam at 100° C, but that expansions after 6 hours of autoclaving at 190° C, were about 1.67 times greater. On these figures, the rate of natural expansion at 18° C increased 500 times by heating to 100° C and 850 times by heating to 190° C. HOSKING and HUEBER, [11], using the expansion after 1 hour autoclaving at 200° C as a datum, found that on the average the rate of expansion varied 2.8 times for each 10° C change in temperature. This relation was found to be valid for the range 100° C to 213° C.

BESSEY, [4], BAKER and JESSOP, [14] have argued that ultimate expansions obtained at autoclaving temperatures are much greater than at ambient conditions because of different reactions involved in the moisture expansion process. BAKER and JESSOP, [14] reported that for a practical cavity wall built in an east-west alignment, it was found after a period of 6 years that the south facing leaves exposed to sunshine expanded by up to 30% more than the shaded north facing leaves.

iv- Temperature of Firing

The temperature of firing the brick in a kiln has a marked effect on the subsequent moisture expansion. Tests reported by HOSKING and HUEBER, [11], HOSKING et al, [12], show that low expansions occur for bodies fired at low temperature while expansions rise to a maximum for units fired between 900° C and 1050° C, after which they decrease for higher firing temperatures. These maximum expansions correspond to the lower end of the industrial firing range and hence most bricks produced show a decrease in expansion with increase in firing temperature. Some clays give quite large changes in moisture expansion for relatively small changes in firing temperature. FREEMAN and SMITH, [10] reported that in the extreme case, a 750° C increase in nominal firing temperature may reduce the average expansion by a factor of as much as 11. Further, the average long term expansion of bricks drawn from the cooler part of the kiln may have expansions 4 times greater than bricks taken from the 60° C hotter part of the same kiln, [11].

v- Humidity

Increasing the relative humidity of the atmosphere to which the bricks are exposed tends to increase the rate of moisture expansion. BAKER and JESSOP, [14] reported that after 1.5 years bricks stored in water have the same expansion as those stored in 100% relative humidity; both expanded 9 times more than bricks stored at 10% relative humidity and 2 times more than those stored at 50% relative humidity. FREEMAN and SMITH, [10] studied eight different clays and found that after 128 days exposure, expansions that had occurred at 90% relative humidity were

20% greater than those at 65% relative humidity. Each clay had been fired at four temperatures covering the range of 800° C to 1025° C.

vi- Cyclic Wetting and Drying

According to BAKER and JESSOP, [14] and HOSKING and HUEBER, [11], the cyclic wetting of bricks at 21° C and drying at 100° C results in far greater expansions than if the bricks were continuously soaked at 21° C. On the other hand, very little difference in expansions of bricks subjected to cyclic soaking and drying at 18° C has been found. This appears to indicate that the cyclic wetting and drying process itself does not affect the expansions of bricks unless the drying is carried out at high temperatures.

vii- Mortar Joints

When built into a wall, the mortar joint restrains movements in the brick and the brick restrains movements in the mortar. Walls tend to expand more in the vertical direction than in the horizontal direction because of less restraint in the former case. An extensive field observation by ANDERSON, [13] indicates that in practical walls the horizontal expansion is reduced about 50% of the unrestrained expansion of individual bricks. BAKER and JESSOP, [14] reported that although the unrestrained expansions of brick and brickwork are identical when both are stored under the same atmospheric conditions, horizontal expansion at the top and bottom of the walls may differ due to different restraints and thermal cycling effects. There seems to be no significant difference in expansions of brickwork incorporating various mortars nor does workmanship appear to be a significant factor.

II.2.1.3 ESTIMATING MOISTURE EXPANSION IN BRICKS

The amount of permanent moisture expansion that takes place under natural exposure conditions varies considerably for different bricks. In practice it is important to have a means of estimating the potential expansion of a brick and, where damage has occurred in a building, the amount of expansion which has taken place already. Three basic ways of assessing the expansion potential of bricks have been reported by BAKER and JESSOP, [14] namely: An index of moisture expansion, short term tests and use of prediction curves. An index of moisture expansion was proposed by FREEMAN and SMITH, [10] and defined as the percentage increase in dimensions occurring between two days and 128 days from kiln hot brick expansion; the first 24 hours' expansion is excluded, since it is large, bears no fixed relationship to subsequent expansion and is of no practical interest since bricks are not used the day after removal from the kiln. The index may range from 0.003% to 0.133%.

A short term test which accelerates moisture expansion may be used to predict the expansion that occurs under natural exposure conditions provided the relationship between the two expansions is known precisely.

Alternatively, prediction is possible provided the relationship is the same for all bricks. It has been proposed by McBURNEY, [50] that the potential moisture expansion of burnt clay units could be taken as that produced by autoclaving the specimen for three hours in steam at 213° C

However, it is difficult to find an autoclave test that satisfies either of the above accelerated criteria.

The third way of assessing expansion potential is to obtain a prediction curve of expansion against time. HOSKING and HUEBER, [11] noted that natural moisture expansion of a brick occurs at an everdecreasing rate with passage of time; this change being approximately exponential, a log-time function gives reasonable agreement with test results.

II.2.1.4 DESIGNING FOR MOISTURE EXPANSION IN BRICKWORK

LAIRD and WICKENS, [8] reported that the method used by both Canadian and American authorities was to specify a moisture expansion of one-half of the thermal expansion and provide for such movement in expansion joints. This is rather a crude allowance as the literature lists many variables. Due to BAKER and JESSOP, [14] the factors to be considered of importance are: moisture expansion properties, age of units and degree of restraint. From this they concluded that only well-fired brick which is at least one year old should be used, and brickwork should be connected to other elements either by flexible anchors, or by using lime rich mortar. According to GRIMM, [44] where horizontal and vertical expansion joints do not adsorb differential movement, thermal expansion of brick masonry alone may induce compressive stresses equal to about one third of the compressive strength of the masonry. From this he concluded that brick masonry should be no stronger than necessary to sustain superimposed loads.

II.2.2 MOISTURE DEFORMATION IN CONCRETE MASONRY

II.2.2.1 GENERAL

Although concrete expands with a gain of moisture and contracts with loss of moisture, the initial drying shrinkage is the greatest concern to the designer; it is dependent upon several factors such as the water cement ratio, composition of the cement mix proportion, aggregate size and curing conditions.

Drying shrinkage has often been considered to be due to surface tension of water in the capillaries. When the material is saturated, surface tension forces are very small, but as moisture

dries from the surface, the surface recedes into the capillaries creating surface tension forces at the menisci. These forces place the material in compression and hence produce a reduction in volume.

There are two major stresses operative when a concrete block is dried:

Firstly, the capillary tension effect which produces a small amount of compression in both cement paste and aggregate. Secondly, a large shrinkage stress due to a loss of inter-layer or chemically-bound water from the cement paste. This water is released by a reaction with carbon-dioxide in the air, called carbonation.

II.2.2.2 REVERSIBLE WETTING AND DRYING EFFECTS

Provided carbonation is avoided, drying shrinkage and wetting expansion are reversible in concrete block. SHIDELER, [16] reported tests on block containing several aggregates that have been steam cured at atmospheric pressure and dried to 50% relative humidity. Some of these units were then exposed to a high concentration of carbon dioxide for four days and finally subjected to alternate cycles of wetting and drying. He found that block which had been carbonated returned to the same length during each period of drying and those that had not been exposed to carbon dioxide showed a greater shrinkage at each cycle; slow carbonation accounted for the continued shrinkage. Uncarbonated block that had not been cycled had about the same shrinkage at the end of test as those that had been wetted and dried.

The magnitude of the reversible shrinkage of concrete products tended to be higher for light-weight aggregate than for dense aggregate, and higher for mixes rich in cement; higher for aerated products than for non-aerated products; and much lower for autoclaved products than for the same products cured in air or steam at atmospheric pressure. BESSEY, [4] gave the following percentage shrinkage for various concrete blocks:

calcium silicate brick autoclaved	
concrete block	0.01 - 0.04%
dense gravel aggregate air cured	0.02 - 0.05%
light-weight aggregate air cured	0.04 - 0.08%
light-weight aggregate autoclaved	0.02 - 0.06%
aerated air cured	0.08 - 0.09%
aerated autoclaved	0.05 - 0.09%
concrete block, dense aggregate, air cured	0.02 - 0.05%

II.2.2.3 CAUSES OF PERMANENT SHRINKAGE

SHIDELER, [17] demonstrated that carbonation of concrete units was accompanied by linear shrinkage. Concrete units predried to relatively low moisture content continued to shrink due to reaction of the hydration product of portland cement with the small amount of carbondioxide on portland cement pastes and mortars. From this he concluded that humidity, size of specimen, concentration of carbondioxide, sequence of drying and curing were important variables with respect to carbonation. SHIDELER, [16] performed drying tests with the presence and absence of carbondioxide. In all cases the presence of carbondioxide resulted in a substantial increase in shrinkage.

ALEXANDER and WARDLAW, [18] suggested that shrinkage was due to the loss of chemically bound water released by the carbonation process. Autoclave curing had produced well crystalized tobermorite which is less vulnerable to reaction with carbondioxide than the disorganized tobermorite gel found in air cured block.

II.2.2.4 CARBONATION SHRINKAGE OF CONCRETE BLOCK

Carbonation is considered to be a reaction between calcium hydroxide released during the hydration of cement and carbondioxide in the atmosphere to produce calcium carbonate and water. BAKER and JESSOP, [21] reported that in a series of tests of concrete block containing several aggregate that had been steam cured at atmospheric pressure, those units that had been carbonated with pure carbondioxide returned to the same length upon drying and wetting cycles. Units which had not been carbonated exhibited progressively greater shrinkage during the wetting and drying cycles. Relatively the same behaviour was observed between batches of autoclaved block that had been carbonated and not carbonated. TOENNIES and SHIDELER, [19] have found that carbonation had a negligible effect on very dry or wet blocks but it had a significant effect on partially dry block. For steam cured block, maximum shrinkage is produced in block that has reached equilibrium in an atmosphere at 50% relative humidity before being subjected to carbondioxide. Carbonation at a relative humidity of 70% to 80% is found to be the most effective for autoclaved block. As temperature increases, the effectiveness of carbonation is reduced in the range of 100° F¹ to 400° F. At 400° F carbonation is ineffective due to lack of moisture. TOENNIES and SHIDELER, [19] stated that an effective plant drying and carbonation cycle must provide conditions to dry the block to a favourable moisture level, maintain the level during treatment and supply adequate carbondioxide for rapid carbonation. Tests were carried out in five plants and it was concluded that the relative humidity in the kiln should be between 15% and 35%, while carbondioxide content should be as high as possible but not less than 1.5% and the temperature should be between 150° F to 212° F.

¹ °C = (°F-32)⁵/₉

II.2.2.5 FACTORS AFFECTING SHRINKAGE OF CONCRETE BLOCKWORK

(i) *Curing Method*

The method of curing concrete block has a marked influence on the amount of subsequent shrinkage. Early tests by WOODWORTH, [20] and ALLAN, [24], indicates that wall panels built with saturated blocks that had been cured in high pressure steam had only about half the shrinkage of panels built with saturated blocks cured in low pressure steam. According to BAKER and JESSOP, [21], curing methods may be broadly classified into four categories, namely: moist air curing, low temperature steam curing at atmospheric pressure, high temperature steam curing at atmospheric pressure and high pressure steam curing (autoclaving).

Most shrinkage comparisons reported in the literature are made between air cured units and units cured in autoclaving conditions. BAKER and JESSOP, [21] reported that autoclaved units have one-half to two-thirds the shrinkage of air cured units.

KALOUSEK et al, [22] carried out a comprehensive comparison of the entire range of curing conditions. They found that curing at atmospheric pressure, in moist air, low temperature steam, or high temperature steam resulted in nearly the same final shrinkage of units. High pressure steam curing reduced shrinkage to about half the shrinkage obtained by other methods.

WASHA, [23] pointed out that the principal reason for autoclaving concrete products is to reduce shrinkage and that the reduction in shrinkage is attributable largely to the more stable type binder in the autoclave process. In properly autoclaved products the binder is essentially a crystalline form of tobermorite. He further pointed out that improperly autoclaved products have been made which had greater shrinkage than products properly cured with steam at atmospheric pressure in moist air. If suitable raw materials are used and autoclaving is carried out at an elevated pressure over 200 psi (1.4 MPa) an xonelite phase is formed in the binder which does not undergo any drying shrinkage.

(ii) *Time of Exposure to Moisture*

No significant case has been found in the literature where shrinkage measurements have been taken on exterior masonry walls exposed to natural conditions.

ALLAN, [24], [25] built walls indoors in an atmosphere which ranged in temperature from 60° to 80° F and in a relative humidity from 32 to 77%. Over a period of 16 months panels built with moist cured block two or seven days after manufacture showed a rapid shrinkage over the first one or two months followed by a more gradual shrinkage which was practically completed after six months. Due to BAKER and JESSOP, [21], it is believed that the reference method—to be discussed in Section II.2.2.7—is the best laboratory method to simulate exterior exposure. Most of the reported similar tests showed an initial rapid shrinkage gradually reducing to stability at one to three months.

(iii) Time of Laying

ALLAN, [24] moist cured concrete masonry units and then allowed them to stand in the air until built into test panels. The panels were made at ages of two, 28 and 90 days after moulding. All panels showed a rapid shrinkage followed by a successively more gradual rate which was practically completed at the end of the 150 day test period. This was confirmed in ALLAN's final report [25] which covered period of one year. The longer the blocks had been allowed to stand in air before laying, the smaller the shrinkage that subsequently occurred in the panels. In general, not much advantage was to be obtained by storing units in air for more than 28 days. BAKER and JESSOP, [21], JESSOP et al, [34], reported that in some tests specimens that had been stored for four months had equilibrium shrinkage (as measured by the reference method of drying) reduced by 25 to 50% compared to those that had been tested at an age of one month.

ALLAN, [24] and WOODWORTH, [20], found that high pressure steam cured units laid in an air dry condition, showed practically no shrinkage. For blocks laid in a saturated condition the results of drying shrinkage tests by the reference method are applicable.

(iv) Humidity and Moisture Content

Early efforts to control shrinkage cracking of masonry limited the moisture content of block at the time of laying to 40%. Extensive tests by KALOUSEK et al, [22] indicated that most of the shrinkage which took place in block when drying from saturation to equilibrium conditions in an atmosphere at 73° F and 25% relative humidity occurred after the block had been reduced to 40% moisture content. Hence limiting the moisture content of block when laid to 40% is an effective shrinkage control.

BAKER and JESSOP, [21], reported that for the autoclaved products the amount of shrinkage is almost proportional to the ambient humidity level: the lower the humidity, the higher is the shrinkage. For air cured products, maximum shrinkage occurs when the products are dried to equilibrium in an atmosphere with 40% to 50% relative humidity. For block cured at atmospheric pressure, carbonation occurs most readily when the block has a moisture content in equilibrium with 40% to 60% relative humidity. Only minor carbonation shrinkage occurs in wet or very dry blocks; on the other hand, autoclaved blocks are relatively immune to carbonation in a normal atmosphere at 50% relative humidity. At higher relative humidity, carbonation shrinkage is significant.

Fig. 2.1 gives the moisture content related to the relative humidity and Fig. 2.2 gives the shrinkage for a change in moisture content. Both Fig. 2.1 and 2.2 show the strong effect of relative humidity on the shrinkage of masonry units.

Fig. 2.3 shows potential shrinkage of block with moisture loss and carbonation during long time exposure to air at various relative humidities.

Fig. 2.1 Moisture contents of light-weight cellular concrete as a function of relative humidity at normal air temperature, Ref. [28].

Fig. 2.2 Shrinkage of light-weight cellular concrete (after 3 days storage in water) to equilibrium in air of room temperature and 43% R.H., Ref. [28]

4

Fig. 2.3 Potential shrinkage of block with moisture loss and carbonation during time of exposure to air at various relative humidities if it is assumed that moisture from the mortar increases the humidity of block initially at 70 to 85% (15% increase) and that of shrinkage A and B could occur, Ref. [28]

(v) Type of Aggregate

SAHLIN [28], BAKER and JESSOP, [21], have reported that the type of aggregate used in the manufacture of concrete block has a significant influence on the shrinkage of the concrete block. Block units made with cinder aggregate shrink about twice as much as those made with sand and gravel aggregate. The range of normal weight and light-weight aggregates includes sand, crushed lime stone, gravel, blast furnace slag, pumice, soria, tuff and cinders. Many light-weight aggregates such as slag, clay, shale, slate, diatomite, fly ash, perlite and vermiculite are prepared by expanding calcining and sintering. Sand and gravel units show the least shrinkage, cinders, expanded shale, slag, and cindered shale show approximately the same shrinkage which is considerably higher than for sand and gravel blocks, blocks made of pumice show the highest shrinkage.

Basically shrinkage occurs in the cement paste, so stiffer aggregates provide greater restraint to shrinkage of the paste and hence tend to reduce the overall shrinkage for products made of those aggregates. Fig. 2.4 shows typical results of shrinkage for units made with different aggregate types.

Fig. 2.4 Shrinkage of concrete units made of different aggregate, moist cured blocks, Ref. [28].

(iv) Mortar Joints

The initial shrinkage that takes place in a concrete block before being built into a wall does not affect the subsequent behaviour of the wall. Walls are built with fresh mortar joints and all of the shrinkage that takes place from the plastic condition onwards influences wall behaviour. In a wall the mortar is restrained by the surrounding block and in turn the blocks are restrained by mortar. As reported by BAKER and JESSOP, [21], JESSOP et al, [34], the use of different mortars has only a minor effect on shrinkage of non-restrained walls and nearly no effect on restrained walls.

HEDSTROM et al, [26] carried out a series of tests on walls built from high shrinkage concrete block units using eight different mortars. Each wall was allowed to dry in an atmosphere at 50% relative humidity while horizontal movement was prevented at the ends of the wall. Cracks occurred in the vertical joints of each wall. Walls built with weaker mortar accommodated about twice as much block shrinkage before cracking as those built with a stronger mortar. According to BAKER and JESSOP, [21] this is attributed to greater creep effects in the weaker mortars. Fig. 2.5 shows typical shrinkage values for some mortar types.

Fig. 2.5 Shrinkage of some mortar types up to 8 days of age (KC is the weight ratio of lime to cement), Ref. [28].

II.3.3.6 SHRINKAGE STRESSES AND STRAINS IN RESTRAINED WALLS

In an experimental investigation reported on by HEDSTROM et al, [26], concrete block masonry walls of granulated slag, three core $200 \times 200 \times 400$ mm blocks and different types of mortar were employed. A first test series dealt with tension under a condition of complete restraint and drying in 50% relative humidity for about 50 days. The specimens subsequently were subjected to direct tension until failure. A second test series dealt with complete restraint and drying from moist condition until failure in 50% relative humidity. The walls were all loaded with a constant 20 psi (0.14 MPa) stress perpendicular to the bed joints to stimulate vertical load in the wall.

The stress-strain diagrams recorded for walls tensioned parallel to the bed joints are shown in Fig. 2.6. The walls were laid in N mortar (made of masonry cement) with 113% flow, 6.4 MPa compressive and 1.2 MPa tensile strength. The wall length is made up of about 2.5% mortar joints and 97.5% block lengths. Out of the total deformation of the wall more than $\frac{2}{3}$ occurred in the form of cracks in the vertical mortar joints.

A third series of tests [26], was run on essentially the same type of walls. In this case the walls were erected and stored for 28 days in 75% relative humidity. Thereafter the walls were prevented from changing length and dried to equilibrium in 50% relative humidity. The resulting tensile stresses were found to be 45 to 65 psi (0.31 to 0.45 MPa) as presented in Table 2.1. The walls did not fail and were subsequently stressed to tensile failure at 0.8 to 0.65 MPa which was slightly higher than for directly loaded specimens reported in Fig. 2.6. The effect of the restraining on the shrinkage of the blocks and the walls as a whole is shown in Fig. 2.7.

Fig. 2.6 Tensile deformations of masonry walls erected and cured at 50% R.H, Ref. [26]

Table 2.1 Loads on Walls at Moisture Equilibrium, Ref. [26].

Type of units	Tensile loads on walls at equilibrium MPa (p s i)	
	Average	Range
Expanded shale-atmospheric steam cured	0.45 (65)	$\pm(5)$
Expanded shale-autoclaved	0.31 (45)	$\pm(5)$
Sand and gravel-atmospheric steam cured	0.45 (65)	0
Sand and gravel autoclaved	0.38 (55)	$\pm(5)$

Fig. 2.7 Shrinkage deformations of walls and blocks, Ref. [26]

Another test series [26] was laid of high shrinkage blocks and stored in 50% relative humidity, thereafter stored for 3 weeks in a moist room, restrained and dried in 50% relative humidity. Walls of eight different mortars were tested; the restraining force increased as the shrinkage progressed up to the point when the tensile bond strength of the vertical joints was exceeded. When the vertical joints failed, the elongations across the joints increased rapidly and continued at a rate greater than if the joints had been closed. All walls in this test series cracked through a vertical joint and across the adjacent blocks rather than along a sagsag line breaking the horizontal joints

in shear. The strength of all mortars (O, N, S, M) was high in this test series probably due to the beneficial effect on the hardening procedure that resulted from the moist storing of the walls.

The walls with the weakest mortars proved to be the strongest since the weakest mortars accommodated larger deformations before failure and tended to decrease stress concentrations. Walls with O mortars produced walls with a tensile strength of about 34% of the block tensile strength and the M mortars only 30%. Since the walls with O mortars had a lower modulus of extension, the joints deformed more and were stronger. These walls accommodated about twice as much shrinkage as the walls with M mortars. The block shrinkage in unrestrained and restrained walls is shown in Fig. 2.8 for four different types of mortars. The free shrinkage from saturation to 50% relative humidity was 0.00067 for these high shrinkage blocks. The wall shrinkage is also indicated in Fig. 2.8. The difference in deformations of blocks and walls are accommodated in the restrained case by stresses and strains in the blocks and joints.

Fig. 2.8 Block shrinkage in unrestrained and restrained walls erected with different types of mortar, Ref. [26].

The joint openings for different equivalent block shrinkages as well as failure elongations for different mortar types are summarized in Fig. 2.9. With no restraints of the blocks from the mortar, the vertical joint openings would be the distance calculated from the shrinkage of the blocks. This gives the straight line in Fig. 2.9. With mortars stronger than the blocks, no joint openings would occur and failure would occur when the extensibility of blocks was reached at 0.0001 to 0.00015 in/in.

The curve of Fig. 2.9 represents the potential shrinkage-joint-opening relation for walls with 100% restraint. The points indicate the failure conditions for each of the eight types of mortar used. Walls with less restraint will give curves below those of Fig. 2.9.

In the related tests by HEDSTROM et al, 1968, [26] show that shrinkage is accommodated to a greater extent by a wall with weak mortar than by a wall with stronger mortar; whether failure occurs depends on the potential shrinkage of the block shown in Fig. 2.9. Mortars containing masonry cement accommodated somewhat greater block shrinkage. A reduction in restraint resulted in greater accommodation of block shrinkage. Some potential block shrinkage (low shrinkage type of blocks installed in dry condition) resulted also in greater accommodation of block shrinkage since a low final shrinkage also implies a low rate of shrinkage, allowing more time to shrinkage stresses to be relaxed by creep in mortar and concrete.

Fig. 2.9 Joint opening effect on shrinkage accommodation, Ref. [26]

II.2.2.7 ESTIMATING SHRINKAGE OF CONCRETE MASONRY

According to BAKER and JESSOP, [21] the major work in this area has been carried out by the American Concrete Institute and the University of Toledo. Four methods of determining drying shrinkage have been used; each method differs mainly by the conditions under which the specimens are dried. The methods are:

i. Reference Method RT-50

Whole blocks dried at 75° F and 50% relative humidity.

ii. Reference Method RT-30

Whole blocks dried at 73° F and 30% relative humidity.

iii. Modified British Method

Face shell prisms cut from whole blocks dried at 73° F and 30% relative humidity.

iv. Rapid method

Whole blocks dried at 220° F to 235° F.

Greater shrinkage occurs with RT-50 drying than with RT-30 drying; this is attributable to carbonation shrinkage which is quite appreciable at 50% relative humidity but small at 30% relative humidity. According to JESSOP et al, [34], the reference method RT-50 is the most satisfactory in representing the practical exposure conditions. The drawback of this test is the long duration, up to 2 months, required to complete the test. The rapid method, which takes two days and has been considered as a practical test for measuring shrinkage, does not lead to a consistent relationship between results obtained on identical block tested by the reference and rapid methods.

The modified British method does correlate with the reference method and only requires the drying of face shell prisms. Both space and time requirements are reduced for reaching equilibrium conditions. Other problems with shrinkage tests appear to be relating test measurements to actual exposure conditions experienced in practice and accounting for varying amounts of carbonation shrinkage under different conditions.

II.2.2.8 DESIGN REQUIREMENTS FOR CONCRETE MASONRY SHRINKAGE

In general, shrinkage is not the only criterion for predicting the cracking tendency of concrete masonry walls, but the fine aggregate units having the greatest shrinkage would be expected to perform poorly.

According to BAKER and JESSOP, [21], and HEDSTROM et al [26] three factors should be considered in designing for moisture movement in concrete block masonry, namely: shrinkage of masonry unit, temperature and moisture conditions at the time of laying, and the degree of restraint.

Although the use of low strength units offers some of the desired flexibility, most of the recommendations seem to be concerned with using low shrinkage units as well as dense aggregate units. High pressure steam cured units and artificially carbonated units are also appreciated. It is also strongly recommended [21], [26], [28] that units are cured at least for a month before laying and kept as dry as possible, that mortar joints be tooled and a weather protective coating be applied to reduce moisture absorption.

Degree of restraint offered for instance by reinforcement, mortar joints and bond beams leads to reduction of shrinkage movements.

II.3 MASONRY THERMAL DEFORMATIONS

II.3.1 COEFFICIENT OF THERMAL EXPANSION

All common building materials contract and expand with variations in temperature. For both restrained and unrestrained conditions, within the elastic range of the used materials these thermal movements are theoretically reversible.

Because of the large variety of building materials used even in the simplest building today, it is necessary to consider the potential thermal movements of various materials which might be used together. Ordinarily thermal movements are estimated from coefficients of thermal expansion based on laboratory tests.

The coefficient of linear thermal expansion for various masonry materials is shown in Table 2.2. There appears to be very little published data on the thermal movements of blockwork and brickwork. According to JESSOP, 1980, [15], it is usually assumed that movements in masonry due to thermal effect are similar in magnitude to those of the individual units although even with a small amount of restraint they may be nearer to half that of the units. Typical values reported for coefficients of thermal expansion range from 3 to 12×10^{-6} per $^{\circ}\text{C}$ for clay bricks and 6 to 13×10^{-6} per $^{\circ}\text{C}$ for concrete blocks, and nearly 13×10^{-6} per $^{\circ}\text{C}$ for 1:3 cement: sand mortar.

Table 2.2 Coefficient of thermal expansion for common building materials.

Material	Coefficient of Thermal Expansion per $^{\circ}\text{C}$
Clay or shale brick masonry	6.50×10^{-6}
Heavy-weight concrete masonry	9.40×10^{-6}
Light-weight concrete masonry	7.70×10^{-6}
Granite	8.50×10^{-6}
Limestone	5.40×10^{-6}
Marble	13.1×10^{-6}
Gravel aggregate concrete	10.8×10^{-6}
Light-weight aggregate concrete	8.10×10^{-6}
Structural steel	11.0×10^{-6}

11.3.2 TEMPERATURE VARIATION

According to JESSOP, [15], and JESSOP et al, [34], temperature differentials for estimating expansion usually are based on the mean wall temperatures. For solid loadbearing walls these may be the temperatures at the centre of the wall. In cavity walls and metal tied walls in skeleton-frame construction, there may be differential thermal movement between the exterior facing and back up and for such construction provisions should be made for maximum thermal movement by considering mean temperature differentials of the exterior wythe.

Very often surface temperatures differ significantly from the surrounding air temperature. It is not uncommon for a masonry wall facing the sun to attain summer temperature as high as 34° C or more in Ottawa area. In addition to the outside conditions to which a building is exposed, one must consider inside temperatures as well as the heat transfer characteristics of the construction between the inside and outside. Horizontal thermal contraction of a unit typically does not induce contraction of the wall but, rather, fine cracks between the mortar and the units, since the mortar unit tensile strength is usually less than the shear strength or frictional resistance to sliding.

Vertical thermal contraction of a unit would typically cause vertical wall contraction. JESSOP, [15] suggested that a design temperature range be adopted in codes for the purpose of computing the magnitude of thermal movement; such a range would be less than the extreme air temperature range. It is probable that a mean range of temperature in the external skin of a wall is nearer to the typical seasonal range, rather than the extreme range, because of the heat capacity of masonry.

The stress induced in a restrained uncracked axial member due to temperature change is equal to the modulus of elasticity multiplied by the coefficient of expansion and by the change in temperature.

II.4 CREEP AND LONG-TERM MASONRY DEFORMATIONS

Creep is the property of a material by which it continues to deform with time under sustained loads. This inelastic deformation increases at a decreasing rate during the time of loading. In reinforced masonry, for example, creep of the masonry materials may increase the stress in the reinforcement to 1.7 times the initial load stress while reducing the stress in the masonry materials to 0.6 times its initial value [15].

The study of creep in masonry has not received as much attention as it should and most of the work has dealt with clay brick masonry. In Canada, where concrete masonry is more important since it is used for loadbearing structures while the use of brick masonry typically is limited to veneer walls, the study of concrete masonry creep should be pursued.

II.4.1 CREEP MECHANISM

Basically, creep in concrete masonry units and in mortar follows the same mechanism as that of concrete. Although creep is separate from shrinkage, it is nonetheless related to it. The internal mechanism of creep (plastic flow) may be due to any one or a combination of the following:

- (a) Crystalline flow in the aggregate and hardened cement paste.
- (b) Plastic flow of the cement paste surrounding the aggregate.
- (c) Closing of internal voids.
- (d) Flow of water out of the cement gel due to external load and drying.

None of these alone seems to entirely account for creep: the gel theory seems to predominate at low-stress level while the crystalline theory predominates at high-stress level.

Creep in concrete, which only occurs to a significant degree under the influence of a sustained load, has its origin in the cement paste, the mechanism is thought to be a combination of moisture transfer which takes place within the gel structure, and collapse of the gel structure due to the load applied. Moisture transfer is most probably the mechanism of shrinkage and swelling of the concrete.

For clay brick, although the structure of the fired clay body is substantially different from the gel structure of the cement paste, such a creep can be explained in terms of moisture movement - which occurs as a result of the cyclic wetting and drying - and collapse of the brick material internal structure due to applied load still seems reasonable.

II.4.2 FACTORS AFFECTING CREEP IN CONCRETE BLOCKS

i- Constituents

Constituents include the composition and fineness of the cement, admixtures, size, grading and mineral content of the aggregate used. The elastic modulus of the aggregate from which the block is made influences the expected creep of the unit. Creep is higher for a low modulus of elasticity aggregate and hence light-weight aggregate units are expected to creep more than normal weight units for the same load.

Proportions such as water content and water-cement ratio also affect creep.

Creep in the units is approximately 70% to 85% of creep in masonry, while creep in mortar joints is much higher than creep in the unit, JESSOP, [15] and JESSOP et al, [34].

ii- Curing Temperature and Humidity

Autoclaved blocks creep less than low pressure cured units for the same reason that such blocks exhibit less shrinkage than non- autoclaved units.

iii- Age and Magnitude of Stress

Creep increases with an increase in the magnitude of the applied loads. Due to JESSOP, [15], for all practical purposes creep is found to be ultimate after 110 days. This has been found to be nearly equal to the magnitude of the instantaneous elastic strain, independent of the level of the applied stress, eccentricity and mortar type. Meanwhile mortars of similar strength give nearly the same creep strain.

II.4.3 MASONRY UNDER LONG-TERM LOADING

As reported by SAHLIN, [28], tests performed on clay brick masonry columns, light-weight cellular concrete masonry columns, and solid concrete brick masonry for a period of 400 days with load (stress level around the permissible stress) and without load in order to study shrinkage and creep for different types of masonry laid with three different types of masonry units. The tests show that the time dependent deformations for clay brick masonry are small, that untreated concrete block masonry has considerable shrinkage, and that a light-weight cellular concrete masonry wall showed considerable response to changes in relative humidity and moisture. The test results are presented in Fig. 2.10 to Fig. 2.14.

Fig. 2.10 Long-time deformations of brick piers

a. Shrinkage of unloaded piers

b. Difference in shortening of unloaded piers

at $\sigma = \sigma_{perm} = 8 \text{ kg/cm}^2$ (0.78 MPa),

ϵ = shortening per mm (-ve = elongation)

I: single brick; II: brick pier, lime mortar; III: brick

pier with mortar of 2 parts lime to 1 part cement to

12 parts sand by volume. Prior to the time 0, the test

specimens were stored for some 4 weeks in air at a relative

humidity of about 70%. Arabic numerals give the designations

of the loaded test specimens, Ref. [28].

Fig. 2.11 Long-time deformations of light-weight concrete piers, a. Shrinkage of non-loaded piers
 b. Difference in shortening between loaded and unloaded piers at $\sigma = \sigma_{perm} = 3 \text{ kg/cm}^2$ (0.3 MPa), Ref. [28].

Fig. 2.12 Long-time deformations of concrete brick piers
 a. Shortening of unloaded piers
 b. Difference in shortening between loaded and unloaded piers at $\sigma = \sigma_{perm} = 6 \text{ kg/cm}^2$ (0.6 MPa), Ref. [28].

Fig. 2.13 Creep of piers calculated in terms of the difference in long time deformation between loaded and unloaded test specimens.

- Brick piers $\sigma = \sigma_{perm} = 8 \text{ kg/cm}^2$ (0.78 MPa)
- Light-weight concrete piers $\sigma = \sigma_{perm} = 3 \text{ kg/cm}^2$ (0.3 MPa)
- Concrete block piers $\sigma = \sigma_{perm} = 6 \text{ kg/cm}^2$ (0.6 MPa),

Ref. [28]

Fig. 2.14 Compression of clay brick masonry walls subjected to stepwise increasing loads. The readings began 1 to 2 h after the walls were laid. Joint thickness 12 mm, modulus of elasticity of bricks 8300 MPa, brick strength 18.6 MPa, the figures on the curves indicate the compressive stress in kg/cm^2 (MPa). The tests were run under laboratory conditions. Lime mortar was used for one wall and lime-cement mortar for the other one, Ref [28].

II.5 MASONRY ELASTIC DEFORMATIONS

Elastic deformations of the masonry assemblage are influenced by the properties of the units and the mortar. The heterogeneous nature of the materials involved makes the prediction of Young's modulus a difficult goal and for this reason a wide variety of empirical and mathematical expressions are reported in the literature.

In the 1978 CSA code [27] this modulus is linearly related to the ultimate failure strength of masonry as: $E = 1000 f'_m$. Although this simple relation is convenient and seems reasonable for concrete masonry, it does not take into account the effect of different factors involved such as the units, mortar and geometrical configuration of the blockwork.

The scatter of the experimental data used to derive this relationship is large due to the fact that f'_m is a function of the ultimate strength of both the units and the mortar. For brick masonry, a value of $700 f'_m$ provides a better fit [15].

II.5.1 ESTIMATING ELASTIC MODULUS

The methods for estimating the elastic modulus of masonry are:

- i- Measuring the strain which corresponds to the applied stress on a prism or panel assemblage and hence calculating the tangent and/or secant modulus.
- ii- Relating the calculated values of the modulus as obtained in (i) to the properties of the masonry units ignoring the influence of the mortar. Such tests have demonstrated that the elastic modulus of masonry can best be correlated to the compressive strength of the unit; this result is similar for brick and concrete masonry.
- iii- Developing theoretical models to predict the elastic modulus for masonry from a knowledge of its components (unit, mortar and grout), material properties as well as geometrical configuration. The objective of such theoretical modeling is to use relatively simple models to predict not only the elastic modulus of masonry but also its creep and shrinkage properties. Values obtained by these models are reported to be within 20% of the experimental values, [15].

II.5.2 ELASTIC MODULUS OF UNITS VERSUS GROUT AND MORTAR

The realization that the grout and mortar have a lower elastic modulus than the block could be used to explain the compression failure pattern of the grouted block specimens. This low elastic modulus in grout or mortar suggests that high lateral strains are developed in them, which in turn induce an outward stress on the confining face of the block unit, the following discussion of the strength of the assembled masonry is quoted from GLANVILLE, [86]:

Although the actual strength of assembled masonry is obviously related to the strength of the component materials, the relationship is not a simple one. The main purpose of the strength testing of the component materials is to insure that the specified quality is being maintained.

When masonry is being constructed, the units are laid on beds of mortar in successive horizontal layers, referred to as courses. The mason normally places the mortar for both the bed and head joints, that is the horizontal and vertical joints respectively, over the face shells only, and not on the cross webs. As a result, a void is created under the webs and, for plain masonry, loading is transferred only through the face shells. Even in grouted masonry, where well-placed grout attains good continuity for the height of each core, it is unlikely that the space below the webs is adequately filled. Under compressive loading, this discontinuity of material leads to stress concentrations and the full potential of the block is never realized. The mortar, on the other hand tends to exhibit an enhanced strength. This is due to the fact that mortar, normally being weaker than the material in the unit, has a lower modulus of elasticity and a higher Poisson's ratio. As a result, mortar tends to strain laterally more than the material in the unit, but being placed in a thin 10 mm layer, it is laterally restrained by the unit. The mortar is then in a triaxial compression and has a higher apparent compressive strength. The lateral compression in the mortar is balanced by lateral tension in the masonry unit, which further reduces the ability of the unit to resist compression. As a consequence of these two factors, that is discontinuities and lateral tension, masonry assemblages have a compressive strength lower than that of the individual units. Although grouting results in increased load-carrying capacity, the value of f'_m obtained for grouted masonry (based on the gross cross-sectional area) is less than that obtained for ungrouted masonry (based on the net area). This is attributable partly to the voids that inevitably exist in grouted masonry, and to the debonding of grout from blocks that may result from grout shrinkage, but more significantly to differences in the stress-strain behaviour of block, grout and mortar.

II.6 NOTE ON FREEZE-THAW ACTION

II.6.1 GENERAL

Decay due to freezing is a serious consequence of the wetting of masonry. Damage done to masonry materials by freezing depends in large measure on their moisture content. When the materials are frozen, the deterioration of masonry materials because of frost action is more rapid and intensive especially in those wall areas where excessive wetting of masonry occurs as for example at the top of a wall. The most important observations relating to frost action as reported by DAVISON and SEREDA, [39], LITVIN, [40], and RITCHIE, [41] are:

1. The severity of mechanical damage is directly proportional to the water content of the porous solid. In a fully saturated state, few, if any, systems can endure even a single freezing and thawing cycle without injury.
2. Physical size of porous solid affects susceptibility; frost resistance improves with reduction in size.
3. Mechanical damage is enhanced with increased cooling rates. Even the most vulnerable system can be taken through freeze-thaw cycles without injury if the freezing rate is very low.
4. Solids with either very high or very low porosity usually have a good service record. Brick and marble respectively, are examples of such materials. Hydrated cement paste with intermediate porosity is usually vulnerable unless special precautions are taken. This is particularly true in the case of high water-cement ratios.
5. Air entrainment, which consists of the addition of a surface-active agent to the plastic mix resulting in the formation of small air bubbles, around which the paste subsequently hardens, has proved to be an excellent method of increasing the frost resistance of cement and concrete.
6. Repeated freezing and thawing under natural conditions usually results in desiccation and accumulation of the formerly pore-held liquid outside of the body (lens formation).
7. Mechanical damage is more severe if the porous solid contains a solution instead of a pure liquid. The use of de-icing salts is detrimental. Again the nature or the extent of damage does not depend on the chemical nature of the solute. The severity of the damage is a function of the solution concentration, the most severe damage occurring at relatively low concentrations, in the 2 to 5 percent range.

II.6.2 DESIGN PRINCIPLES TO IMPROVE FREEZE-THAW DURABILITY

In order to avoid mechanical damage due to frost action, the moisture content should be such that the amount of excess water generated in unit time is less than the quantity lost by the porous solid to the exterior in the same period. This condition will exist if the total moisture content of the body is low, in which case no excess water is produced, or failing that, the permeability is high

and the cooling rate is low, or permeability low but thickness of elements small (spacing between air voids).

Sources of water in porous building materials are precipitation, driving rain and melting snow. There are various ways to minimize the ingress of water by careful design. Vertical walls can be protected with roof overhangs, eavestroughing, and cladding. Horizontal surfaces always should be provided with a slope, crown, or camber to promote drainage and avoid the formation of water pools.

Another, often overlooked, source of water is condensation from the vapour phase. In winter time, walls of enclosures are usually the coolest region of the interior space and unless effective vapour and air barriers are installed, water vapour condenses in the walls. As a rule no reasonable effort to avoid high water content during service should be spared in the design stage. With relatively modest expenditure the occurrence of serious and costly problems usually can be prevented.

II.7 CRACKING AND CRACKING CONTROL

II.7.1 CRACKING

Having reviewed the different types of deformations that might affect the behaviour of masonry materials and assemblage, we are now able to study such effects in more rational ways by looking at the direct effect of those deformations or at cracks.

The satisfactory performance of masonry walls depends upon three factors: the materials specified for construction, the design utilizing the materials, and the construction used in carrying out the design. When one or more of the above requirements are neglected there is the possibility that the masonry wall will not perform satisfactorily.

Cracking is the failure which probably occurs most often. It is necessary to understand the causes of stress in order to carry out a design which will accommodate the anticipated movements. Cracks develop in many different ways but there are typical modes and characteristics; often the type and magnitude of cracking indicate the cause of cracks.

Cracking can be an annoying phenomenon in masonry structures and their main cause is the differential movement of different building parts or materials. Differences in stress level, settlement, deflection and shrinkage of the roof slab as well as the fluctuations in climatic conditions are principal causes of cracks. Cracks may be of different sizes and patterns. Fig. 2.15 shows the most common types of cracks in masonry walls. Table 2.3 summarizes the principal causes and effects of cracking.

The review presented in the following sections for different crack modes and their probable cause is based on the practical experience reported in the literature [7], [28], [29], [30], [32], [33], [43] and [71].

Fig. 2.15 Common types of cracking in masonry walls, Ref. [51]

11.7.2 CAUSES OF CRACKING IN MASONRY WALLS

1. Slab (Roof) Deflections

When a floor slab resting on a wall is loaded, the end rotation of the slab can develop cracks at the wall slab interface. This type of cracking is more visible in the external walls although it may occur at internal walls as well. This crack is setup by the upward lift tensile force in the roof slab due to the rotation of the end of the slab.

Cracking risk decreases as the force in the wall (and the restraining moment on the slab end) increases. Theoretically cracks will be formed if the induced tensile forces at the outer face of masonry wall exceeds the permissible tensile stress of the wall. This type of crack can only be accommodated by providing tensile reinforcement sufficient to handle the tensile stress or by separating the slab from the wall. Another way is to reduce the slab deflection by increasing its stiffness, but zero deflection may not be practically possible.

2. Differential Strain from Stress and Temperature Changes

The differential strains can occur as a result of different expansion characteristics of two walls, or a wall and a column with different Young's modulus or might be due to the difference in stress level for the same material walls. Moreover external walls are subjected to fluctuations of temperature and hence they tend to deform differently than internal walls, causing cracks in the connecting walls. This action may also happen in cavity walls connected by metal ties bridging between the two wythes. To avoid this type of cracking caution should be exercised when choosing materials and stress levels for wall in higher buildings at least. A further summary is given in Table

2.4.

3. Roof Expansion and Contraction

Generally the foundation of a building is considered to be at a fixed temperature while the roof is exposed to daily and seasonal temperature fluctuations. Due to geometrical features and orientation of the building some walls will receive more direct sun rays than the others and hence deform to different amounts. The outcome of this differential movement is to induce stresses in the wall section due to relative movements of the roof and the wall. Long horizontal cracks tend to form when these stresses exceed the permissible strength of the blockwork. This of course will occur only if no provisions were made for an independent movement to take place.

4. Creep and Shrinkage Cracking

A masonry wall is normally restrained to some extent along its edges or at certain points. Differential shrinkage between the masonry and the restraining media will therefore build up stresses in the masonry. Due to creep such stresses are relieved to some extent in certain cases. In other cases the creep can be a source of differential movements and accompanying stresses.

As soon as the bond strength between the vertical mortar joints and the block is exceeded, cracks will open in these joints. The cracks are typically 0.025 to 0.076 mm wide prior to failure, according to SAHLIN, [28]. The wall is divided into two or more parts by continuous cracks of the wall. The restraint can be reduced by introduction of control joints. Cracking may develop in shear through the vertical and horizontal joints or by bond tension failure in the vertical joints and tensile failure in the blocks along the line of the vertical joints. According to SAHLIN, [28], two core block could provide somewhat higher resistance to the latter type of cracking. Table 2.5 shows a summary of drying shrinkage cracking and how it is prevented.

5. Poor Details and Improper Construction

Poor construction as well as insufficient details are another common reasons for cracking development and may include:

- a - Wrong location of control joint(s).
- b - Design of the drain.
- c - Alignment of windows.
- d - Placing of steel plates above an opening.
- e - Location of splices.

Usually control joints are laid up in mortar first, and once the mortar has hardened it is raked out to a depth of about 20 mm. The remaining mortar then provides a backing for the caulking. Quite frequently it appeared as though the mortar was not raked out of the control joint, thus eliminating benefits derived from the joint.

To avoid spalling that may occur as a result of the freeze-thaw action care should be taken when selecting the run-off drain for the roof. If the water from the drain is allowed to fall back into the wall, the freeze-thaw effect will cause spalling.

A window opening(s) has two vertical planes of weakness on either side of the opening. For a window opening above another opening, the sides of the opening should be aligned vertically if possible and if not, one side should be aligned.

To avoid spalling due to an eccentricity of the load, caution should be exercised when placing the steel plate above an opening. If the mortar between the plate and the block is not deep enough the load may be applied to one portion of the block. The stresses caused by the eccentricity of the load may cause cracking of the supporting blocks below the steel plate. Some designers

will utilize part of the opening for a movement joint, but this procedure will lead to problems in practice. Above the opening lintel beams or soldier courses carry the load from the blocks above the supporting block for it to act properly. Since the soldier course bears on the supporting block it is hard to establish a gap between the beam and the block.

Joint reinforcement is used so that there will be many fine cracks as opposed to one large crack. Proper splicing of the joint reinforcement should be insured so that a potential crack does not form. Table 2.6 presents a summary of cracking due to associated elements.

6. Running Bond Versus Stacked Bond

The type of bond used will determine the strength of a concrete block wall. This was reflected in Table 5, in the 1978 CSA code, [27], for non-reinforced block walls. The tension parallel to bed joint is twice that normal to the bed joints.

Joint reinforcement at 400 mm intervals gives the same strength for two bonding patterns which is an increase of 20% for the running bond and 120% for the stacked bond [29].

Joint reinforcement at 200 mm intervals show an increase of 60% for running bond and a four-fold increase for stacked bond [29].

7. Load Bearing Versus Non-Load Bearing

The designer must make a choice, whether to use loadbearing masonry walls or not. A loadbearing wall will tend to have a clamping action making it harder for a crack to form in the bed joint by increasing the shear friction between the unit and the mortar joint.

8. Foundations

When masonry walls are built on concrete foundations that extend above the grade, thermal expansion of the wall (masonry) may work against the drying shrinkage of the concrete causing extension of the masonry wall beyond the foundation. When the masonry wall contracts with lowering of temperature the tensile strength of masonry is not sufficient to move the masonry wall back with it thereby causing cracking in the masonry near corners. Table 2.7 presents a summary of cracking due to foundation movement.

9. Settlement

Foundation settlement is one of the common reasons for cracking, especially in soft soils such as leda clay (Ottawa area). Very often detrimental settlement occurs in smaller buildings where foundations are designed by guess. Settlement cracks are usually larger at the top diminishing to a hairline crack at the bottom or vice versa, depending upon the relative direction of the settlement and its location with respect to wall length. It should be noted that it is the differential settlement rather than the total settlement which is the trouble maker.

A further summary for miscellaneous causes of cracking other than the above-mentioned is presented in Table 2.8.

Reference [51] from which Tables 2.4 to 2.8 are quoted is a useful document in which a practice oriented study of cracking problem is presented. The report summarizes different modes of cracking and their expected reasoning as well as suggested remedies to overcome such cracking. The reasonings as well as the remedies seem to be based on practical experience and engineering judgement rather than analytical or experimental evidence. The report was written for the Australian conditions and hence, it may not be directly applicable to Canadian practice.

Table 2.3 Summary of movements and their principal causes, Ref. [47]

Cause	Effect	Duration	Examples of materials affected
1. Temperature changes	Expansion and contraction	Intermittent diurnal, seasonal	All
2. Moisture content changes			
a) Drying	Shrinkage	Principally short term, due to loss of initial moisture	Mortar, concrete, sand lime bricks, unseasoned timber
b) Wetting	Expansion	Short term, due to initial take up of moisture	Ceramic products
c) Drying alternating with wetting	Expansion and contraction	Seasonal	Poorly protected joinery, shrinkable clay soils
3. Other physical changes:			
a) Loss of volatiles	Contraction	Short term or long term	Mastics
b) Ice or crystalline salt formation	Expansion (in building materials); frost heave	Intermittent, dependent on weather conditions	Porous natural stones and other building materials; soils
4. Loading			
a) On structure			
i) Dead and imposed loading within design limits	Normally insignificant		
ii) Structural over loading	Excessive deflection and distortion		
b) On ground	Settlement	Extent of settlement varies with seasons	Silts and peaty ground particularly susceptible
5. Soil movements, e.g. mining subsidence, swallow holes, landslips, soil creep, earthquakes	Settlement		
6. Vibration from traffic, machinery, sonic booms			Authenticated cases of damage are rare
7. Chemical changes:			
a) Corrosion	Expansion	Continuous	a) Metals
b) Sulphate attack	Expansion	Continuous	b) Portland cement and hydraulic lime products, concrete and mortar
c) Carbonation	Shrinkage	Continuous	c) Porous Portland cement products, lightweight concrete, asbestos cement

Table 2.4 Cracking due to expansion of clay brickwork, Ref. [51]

Table 2.5 Cracking due to drying shrinkage, Ref. [51]

Table 2.6 Cracking due to movement of associated elements, Ref. [51]

Table 2.7 Cracking due to movement of foundations, Ref. [51]

Table 2.8 Cracking due to miscellaneous causes, Ref. [51]

CAUSES							
Rusting of embedded steel	Spread of pitched roofs	Bulging of retaining walls	Vertical bonding of load-bearing walls	Overloading of piers	Intentional closure of cavities	Expansion of timber flooring	Thermal changes (rare)
SYMPTOMS							
Horizontal or stepped cracking and spalling from point of embedment	Horizontal cracking some distance below the ceiling on the internal face of the external wall	Horizontal cracking associated with noticeable bulging	Vertical cracks through perpend and bricks beneath point of application of load	Horizontal cracks where cavity is bridged	Horizontal cracks and outwards movements of walls parallel to direction of flooring. Movement of mouldings fixed to floor	Similar to those for the expansion of clay brickwork	
REMEDIES							
Remove the cause and replace with galvanised steel, reinforced concrete or non-ferrous metal.	Strengthen eaves joint, and improve strutting.	Shore up, cut back retained material, improve drainage and hand pack back of wall with heavy self-supporting material.	Improve lateral support by wall anchors, straps or ties, otherwise demolish and rebuild wall.	Demolish piers and rebuild with adequate section. Alternatively use steel or reinforced concrete column.	Mask crack or conceal with V-joint.	Relieve pressure of flooring on wall. Improve underfloor ventilation, or if this is not possible, cover surface of ground under floor with an impermeable membrane.	Refer to Table 2.3

II.7.3 A SUMMARY OF CRACKING CASE HISTORIES

Only a few surveys have been reported in the literature concerning cracking. References [7], [28], [29] and [51] are among those who have reported some of that surveys. The following is an overall view of the modes of crack formation and their possible reasons as inspired by these case studies. It is worth noting here that most of the information and findings from these case histories has been covered already under the previous sections.

Type 1: Horizontal Cracks at the Uppermost Slab

This type of crack is due to differential movement between the concrete roof slab and the external wall. Part of this differential movement comes from the high shrinkage of the concrete. The suggested remedy is to free the slab from the wall completely, and the cracking that will occur due to the sliding in the re-entrant corner must be covered by repainting or by moulding after the main portion of the shrinkage takes place.

Type 2: Horizontal Cracks at Intermediate Floor Slabs

The causes of this type of crack is either similar to Type 1 or is due to the slab deflection effect. To prevent this type of cracking, large openings (windows, etc.) should be avoided especially at the corners. Separation of the wall from the slab by means of a flexible material may be useful.

Type 3: Cracks in the Foundation Wall

This crack usually occurs at the reduced sections of the foundation walls. The probable causes are poor foundation, section changes and stress concentrations eventually in combination with temperature and moisture gradients. Proper design, better survey of the building site and use of low shrinkage concrete are believed to prevent this type of cracking.

Type 4: Vertical Cracks at the Building Corners

When the concrete slab shrinks, the external walls are relatively free to follow the slab between the corners, but at the corners such movement is prevented by in-plane forces in the external walls, resulting in deflection of the external walls as shown in dotted lines in Fig. 2.16. As a result high bending moments occur at the corners of the building and since the section of the external wall is usually weakened by the bond pattern these moments cause a vertical crack one unit away from the corner. The apparent solution is the use of reinforcement.

Fig. 2.16 Assumed mechanism of Type 4 cracking, Ref. [28].

Type 5: Cracks Between Roof and Wall

As already has been discussed, this type of crack is due to the differential movement between the roof and the wall. Regardless of the material used in the roof the thermal movements of the roof structure are much higher than that of the external walls, which is a good reason for permitting the roof to move independently from the wall whenever possible

Type 6: Cracks Around Entrances

This type of crack appears at the lower part of the building, around the entrances. These cracks are probably caused by section changes which occur in the wall and slab at the entrances of a building. Since the entrance openings cause the weakening of the foundation structure, the internal wall and slab. Low shrinkage concrete and a suitable reinforcement and/or expansion joints are possible solutions.

Type 7: Cracks Around Balconies

This type of cracking has the same cause as of Type 6. In addition the movements of the protruding concrete slabs which are unshielded against temperature variations and which therefore have daily and yearly climatic movements outside the building may cause such cracks. Protruding balcony should be jointed to the rest of the building by, for example, bitumenous or rubber material.

Type 8: Cracks in the Mortar Joints

The cause of this type is mainly the successive expansion and contraction of the wall due to thermal and moisture movements. Expansion joints and reinforcement are possible remedies.

Type 9: Vertical Cracks Below Window Openings

These cracks follow the horizontal or vertical joints below the window. The probable causes of this type are the foundation movement, the difference between the very high load in the parts of the wall on the side of the window, and the absence of load below the window which could cause excessive stresses and cracking. This crack could be avoided by a better foundation design eventually in the combination with the reinforcement in one of the joints immediately below the window

Type 10: Horizontal Cracks at Building Corners

This type of crack is caused by the upward lift of the slab corners, as has been discussed earlier. Only reinforcement or separation of the wall from the slab can minimize such cracks.

Type 11: Cracking at Slab Level

This type of crack occurs when the thin masonry units outside the slab edges are poorly bedded in the mortar joints and heavily loaded from above or could be due to the disturbance of masonry unit(s) during the pouring of the concrete slab.

11.7.4 CONTROL OF HORIZONTAL MOVEMENTS

As has already been discussed in the previous section, walls and all materials that make up a building are in a constant state of movement due to elastic, creep, shrinkage, and thermal deformations, acting together or individually. There are two techniques reported in the literature for accommodating such movements and controlling cracking. Those techniques are:

- 1- to provide reinforcement, and
- 2- to provide control joints.

1. Horizontal Reinforcements

This was developed to help control cracking in walls, and all it does is distribute the stresses so that there will be a number of fine cracks as opposed to one or two large ones. It may be in

the form of reinforced collar joints or bond beams. The joint reinforcement is placed at vertical distances from 200 to 800 mm; having a minimum cover of 10 mm and the bars should overlap at the ends by at least 150 mm. One should note that the use of minimum reinforcement in masonry may not insure crack control under moisture changes and thermal deformation.

2. Control Joints

Control joints are vertical joints built into a wall to relieve stresses, that may be either tensile or compressive in nature, by reducing restraint and permitting movement to take place.

In clay brick walls the dominant movement is a slight expansion which, in long walls, can create excessive stresses and cracking in the brickwork. Hence the expansion in a long wall should be controlled. A common method of control is the inclusion in the walls of expansion joints which provide space into which the expanding masonry can move. Vertical expansion joints are used to accommodate horizontal movements in masonry walls, and horizontal expansion joints are used to adsorb vertical movements between walls and frames of buildings.

In walls of concrete or sand-lime masonry the dominant movement is a slight contraction which, if not controlled, can result in excessive stresses and cracking even in short walls. A common way of control is the provision of joints in walls, such joints are able to open when the adjoining masonry contracts. The usual types of expansion joints can open as well as close, and thus function also as contraction joints.

There have been many attempts to come up with a specific answer to the question of where the control joint(s) should be located, question that is further complicated by the nature of thermal and moisture stresses in masonry walls which involves many separate phases, and variable factors and it does not lend itself to a neat mathematical solution.

COPELAND, [73], based on simplified assumptions and a limited number of experimental data, proposed two methods of locating control joints in blank walls which in general tend to be conservative solutions. HENDRY and NOURANI, [74], proposed a theoretical solution to the problem of locating the control joints. Based on the assumption that the roof temperature is higher than the wall temperature, they arrived at theoretical curves by solving the compatibility and equilibrium equations at the roof-wall interface. The theoretical nature of the method and the lack of analytical and experimental support limit the use of the method to serve as a guide only.

The recommendations provided by the American Concrete Institute [85], is a useful guidelines and probably the most widely used in North America. The report is dealing with concrete masonry crack control procedures.

Control joints should be incorporated into a wall in locations at which stress concentrations might occur: at locations where there are abrupt changes in wall height or in wall cross-section. Joints may also be located at pilasters and at pipe or duct chases, above joints in the foundation or floor, below joints in the roof or floors bearing on the wall, at a distance not over one half the

allowable joint spacing from bonded intersections or corner, and at one or both sides of all door or window openings depending on its size. Table 2.9 shows typical spacings of control joints.

A control joint must not only allow for movement but must also tie the two sections together and provide a water-tight seal.

Work that has been reported in this area [29], [30], [32], [33], [46], [47] does not make a clear distinction between control joints in brickwork and blockwork, which might be due to similarity of the nature of movements in the two types of masonry walls although their magnitude and direction may be different.

References [30] and [33] suggested a general guide for control joint location and materials, and this is presented in Fig. 2.17 to Fig. 2.23.

Table 2.9 Typical Spacing of Control Joints, Ref. [32]

Joint Reinforcing Spacing mm	Maximum Control panel H/L	Spacing of Joints panel length m
none	0.5	12
600	0.4	13.5
400	0.33	15
2000	0.25	18

II.8 DESIGN CONSIDERATIONS FOR HORIZONTAL MOVEMENTS

Due to the wide variety of the already mentioned factors involved in the generation of horizontal movements in masonry wall; namely elastic, creep, shrinkage, and thermal effects, it became almost impossible to write down general rules or specifications that are universally true and applicable. The reported literature describes a set of rules and cautions that are based more on art than science, in which experience was the dominating factor. The considerations presented herein are taken from BESSEY [4].

11.8.1 GENERAL DESIGN

It is important that dimensional change characteristics of the materials to be used be considered at the same time as the general design of the building. The designer may approach this in two ways, one being to modify design to suit the materials, the other to select materials carefully to fit the design. In practice however, it is usually necessary because of economic considerations and other factors to compromise between these two approaches and to use those materials which produce minimum movement, or to design the masonry wall to accommodate movement.

11.8.2 LOCATION AND FREQUENCY OF MOVEMENT JOINTS

The positioning and frequency of vertical movement joints must generally be a compromise between the technical need to reduce the possibility of cracking to the minimum on the one hand and the cost and the aesthetic considerations on the other. It is not possible to lay down strict rules or write a specification for the spacing and location of joints to suit all structures but some general guidance is possible:

- (1) Clay brickwork should have movement joints at about 12 meter centers. In a cavity wall, having clay on one wythe and concrete units in the other, this interval should be reduced to between 6 and 7.5 meters.
- (2) Plain concrete block masonry walls should have movement joints at not more than 6 metre intervals. While the frequency of movement joints suggested afford useful guidance, the following modifying factors need to be taken into consideration in locating movement joints in particular circumstances.
 - i. In thin walls less than 200 mm thick with no openings, connected to supporting piers or columns at intervals, movement joints may need to be at shorter intervals than in thick walls, but this will depend on the frequency of the piers and the bonding. It may be convenient to form a movement joint at the side of each pier or at alternate piers.
 - ii. In thicker walls, since the tensile failing load is higher, it is possible to lengthen distances between joints. Similarly, high walls will tolerate greater lengths than short walls.
 - iii. When openings are present the frequency of such openings and the heights of brick or blockwork above and below them, in relation to the total height, will influence the need for joints. Where the total section of wall is much reduced by the openings, the stress concentration will be great and movement joints need to be more frequent.
 - iv. Where linear and outer wythes of cavity walls consist of materials of different dimensional change characteristics (eg clay brick and concrete block), and are rigidly connected at the end or at the openings, then the frequency of movement joint needed is determined by the algebraic sum of the maximum probable movements. Thus if one is likely to expand

slightly and the other to shrink slightly, the movement joint will need to be more frequent than if both were of the same material. Where possible, rigid connections of two different materials in this way should be avoided.

- v. Where there are features which can be used to conceal movement joints, the positioning should be adjusted to take advantage of this possibility even if this necessitates closer intervals than would otherwise be required.
- vi. Where brick or blockwork is returned about steel or reinforced concrete frame members and there is no clearance left between the brickwork and the frame member, it may impose restraint or may transmit movements of the frame to the brickwork. It is preferable to isolate the brickwork from columns either by a space or by resilient membrane such as bitumen, bitumenous silt or a joint filler. Where this is not done, provision made for movement should take the imposed restraint into account.
- vii. Restraint at top and bottom edges of brick or blockwork is reduced by the presence of dam-proof coursing, which reduces shear resistance. This fact may reduce the need for vertical movement joints or modify the desirable positioning of them.

Fig. 2.17 Expansion joint fillers, Ref. [33]

Fig. 2.18 Typical expansion joint placement, Ref. [33]

Fig. 2.19 Expansion joints at offsets and juncture, Ref. [33]

Fig 2 20 Expansion joints in straight walls for location of joints see Fig 2 19, Ref [33]

Fig. 2 21 Expansion joints near corners in skeleton-frame buildings, Ref [33]

Fig. 2 22 Horizontal expansion joints at shelf angle, Ref [33]

Fig. 2.23 Enlarged detail A

II.9 CONCLUDING REMARKS

- A significant amount of research has been carried out with most of it being devoted to deformations of clay brick and concrete block units
- The precautions to be taken in design should be related to the properties of the specific materials being used. Thus whereas with clay bricks it may be in some instances desirable to make provision against expansion, with concrete and calcium silicate materials some provision is usually required against shrinkage. With all types of bricks and blocks, provision must be made against thermal movement (either expansion or contraction)
- The precautions in design which may be useful in preventing cracking include the provision of movement joints, use of reinforcement to assist in distribution of stresses, and location of openings and features so that movement does not lead to accumulation of stresses at points of weakness that could lead to conspicuous crack formation
- The importance of the various effects varies greatly in different countries and indeed even in different parts of countries. Climatic factors and building-techniques locally give greater importance to one or other effect, also the raw materials available may make one type of block or brick suffer from a given defect very much in one area and very little in another. It is necessary to have a knowledge of all these factors in relation to building in a given area, if knowledge of the defects that are prevalent in brickwork and blockwork in that area is also available, this may also assist in avoidance of further trouble.
- There is insufficient data available for calculation of the amount of the unrestrained movement that can be accommodated without risk of cracking. Whilst the elasticity and the tensile and the compressive strength of masonry panels may be determined, bond and shear strength, creep effects and the distribution of stresses induced by shrinkage and expansion can not be easily assessed and applied quantitatively to actual structural design.
- One of the major drawbacks of the available literature is the absence of extensively carried out case studies and surveys that relates the cracking problem to the local climatic conditions. The importance of including the climatic conditions as a factor in studying deformations arise from the fact that excluding settlement, moisture and thermal deformations are the common reason for cracking. There is no evidence in the available literature about neither the probability of occurrence of the different types of deformations, nor how the thermal and moisture variations along the wall height might be.
- A rational model needs to be developed and analyzed by means of numerical methods

CHAPTER III

FINITE ELEMENT ANALYSIS

III.1 GENERAL

In the last two decades there has been a marked increase in structural masonry research, wall research has concentrated on studies of in-plane behaviour of masonry with particular emphasis on the behaviour of walls subjected to varying combinations of shear and compressive forces. This research has involved either tests on complete wall panels at full or model scale, or tests on small samples of masonry to simulate the local stress state in large panels.

The principal task of such research was to produce test information which could form the basis for design rules to be incorporated in codes of practice. The resulting design rules, hence, tend to be based on semi-empirical relationships established from tests on structural elements.

Very little emphasis has been placed on the development of a fundamental theory of failure for masonry which could be applied to any case of in-plane loading. Structural masonry is a composite material whose behaviour depends on the properties of the constituents (brick or block and mortar). In general, factors which influence the strength and serviceability may be classified as:

- (i) Those related to physical properties of basic blocks, such as dimensions of the units, density, initial rate of absorption (IRA), degree of perforations, thermal properties and dimensional uniformity.
- (ii) Structural properties, like compressive strength, modulus of elasticity, modulus of rupture, creep of mortar and units used.
- (iii) Workmanship, bonding and bed joint thickness.

All the a forementioned factors can influence the strength and/or serviceability to a significant degree.

III.2 SUMMARY OF PREVIOUS WORK

Research has by and large been devoted to the study of elastic stress behaviour although basic structural behaviour warrants elasto-plastic study. Isotropic elastic behaviour has usually been assumed to simplify the problem, and the influence of mortar joints as planes of weakness has usually been ignored. PAGE, [54] and [56] has compared the deformations he obtained by using a finite element elasto-plastic model he suggested with those by using the conventional finite element approach and with experimental results. The comparison has shown that good agreement is obtained for low stress levels and the error introduced by an isotropic assumption in the conventional model increased as the level of stress increased; from this he concluded that the simplified isotropic behaviour assumption may be satisfactory in predicting deformations at relatively low stresses in the working stress range, but cannot be expected to be adequate at higher stress levels when extensive stress redistribution will occur. This distribution is due to non-linear material behaviour (predominantly in mortar joints) and failure in localized areas due to loss of bond between mortar and units.

The model used by PAGE was an analogy drawn with jointed rock, a problem which has been extensively investigated in the geological and geotechnical literature [60], [61], [67].

NGO and SCORDELIS, [58] in their modeling of reinforced concrete beams was the first to apply the idea of modeling two materials simultaneously in finite element analysis.

GANJU, [53] might have been the first to adopt the elasto-plastic analysis of brickwork, his model as well was inspired by jointed rock analogy.

A theoretical model has been used by AMENY et al [66] to predict the elastic behaviour of masonry and has been extended by SHRIVE and ENGLAND, [59] to incorporate the creep and shrinkage of units and mortar. The model used to estimate the overall elastic and time dependent deformational behaviour of masonry from the properties of the constituent units and mortar

The work performed by PAGE [56] and PAGE et al, [55] although it is a step forward toward better prediction of masonry behaviour and rationality of analysis methods, suffers, however, from its dependence on the properties and characteristics of the constituents, a task which seems difficult to limit, let alone quantitatively determine, even within a limited place and time. The model could not predict the first crack of the assemblage since its failure criterion is governed by the predominant failure of joints as defined by 3 regions on the ultimate normal stress versus ultimate shear stress experimental curve. Hence final failure cannot be predicted since this will involve both brick and mortar.

The model suggested by GANJU, [53] in which brickwork is modeled as a strain-hardening elasto-plastic material, uses a yield criterion based on the generalized Mohr-Coulomb rule. Failure is defined by limiting the value of strain while the effect of mortar joints as planes of weakness is ignored.

The foregoing discussion presents the limited amount of research that has been carried out to model brickwork masonry for in-plane loading conditions.

For block masonry, very little has been reported. PAGE et al, [55], referred to what they called an extensive analytical and experimental program at the University of California that produced a finite element model capable of reproducing the behaviour of reinforced concrete block masonry walls. The model apparently incorporates a non-linear material which accounts for masonry cracking and the effects of reinforcing steel; it also appears to allow for the pre- and post-fracture behaviour of the joints. With regard to reinforced brick masonry, a model that has the advantages of University of California model [69], [70], has not yet been developed. A model capable of treating composite masonry differential deformations has been suggested by ANAND, and YOUNG, [62].

With regard to cracking (serviceability) problems in masonry walls, the research quoted in the literature seems either to ignore these problems entirely or to address them in a superficial manner.

III.3 TEMPERATURE DEFORMATIONS

When the temperature of a structure varies there is a tendency to produce changes in the shape of the structure. In order to obtain formulas for deformation due to temperature, we will consider the rod shown in Fig. 3.1.(a). A uniform temperature change throughout the rod results in an increase in the length of the rod by the amount:

$$\Delta L = \alpha L \Delta T \quad (3-1)$$

Where:

ΔL = change in length (positive denotes elongation)

α = coefficient of thermal expansion of the rod material

ΔT = change in temperature (positive in case of temperature rise)

Fig. 3.1 Thermal deformation in a rod.

In addition all other dimensions of the bar will be changed proportionately, but only the change in the length will be considered important in this case. The deformation of the element is shown in Fig. 3.1.(b) and appears to be of the same type as that caused by an axial force. This deformation is given by equation (3-1) from which the temperature strain is:

$$\epsilon_t = \alpha \Delta T \quad (3-2)$$

Equation (3-2) will be used later to convert strains that are not due to thermal effect to an equivalent fall or rise in temperature.

III.4 FINITE ELEMENT FORMULATION OF THE INITIAL STRAIN PROBLEM

In order to make it possible for the finite element method to solve for strains $\{\epsilon_0\}$ that are not due to applied loads, such as thermal strains, creep and shrinkage, we will solve the problem without reference to a particular element, thus arriving at a general solution that could be applied to any finite element. While going through this process we will add another condition and that is that we will make provision for strains which are not the result of stress. If an element, irrespective of its shape, degrees of freedom or number of components in its stress and strain vectors, has a strain which is not related to load, that is the strain would exist if there were no constraints on the element and it could freely change size (as a result of temperature change or other reasons), then the strain will cause certain displacements which might be significant and should not be ignored. When the force $\{F\}$ is applied, additional displacements will occur giving a total displacement represented by a displacement function $\{u\}$. Total strains can be found from the displacement function by performing certain differential operations given in the operator matrix $[\Delta]$, that is

$$\{\epsilon\} = \{\Delta\}\{u\} \quad (3-3)$$

But elastic strain is given by $\epsilon - \epsilon_0$ and hence the stress is

$$\{\sigma\} = [D]\{\epsilon - \epsilon_0\} \quad (3-4)$$

Where $[D]$ is the elastic property matrix

The displacement functions $\{u\}$ are polynomials and can be written as

$$\{u\} = [P]\{\rho\} \quad (3-5)$$

where $[P]$ contains polynomial terms and $\{\rho\}$ contains the coefficients of the polynomial function. Since the displacement function must give the displacements at the nodes, δ , then when the coordinates of the nodes are substituted into $[P]$ we get,

$$\{\delta\} = [A]\{\rho\} \quad (3-6)$$

where $[A]$ comes from $[P]$ evaluated at the nodes. Solving (3-6) gives the coefficients in the polynomial:

$$\{\rho\} = [A^{-1}]\{\delta\}$$

Substituting this into (3-5) and then into (3-3) gives;

$$\{\epsilon\} = [\Delta][P][A^{-1}]\{\delta\} \quad (3-7)$$

The operations of $[\Delta]$ can be performed on $[P]$ giving a new matrix $[B]$

$$[B] = [\Delta][P] \quad (3-8)$$

Putting (3-8) into (3-7) gives,

$$\{\epsilon\} = [B][A^{-1}]\{\delta\} \quad (3-9)$$

Which if substituted into (3-4) gives:

$$\{\sigma\} = [D][B][A^{-1}]\{\delta\} - [D]\{\epsilon_0\} \quad (3-10)$$

Now we have an element that has its nodes displaced by δ and is held in this position by some, as yet unknown, force $\{f\}$. From this state let us give the element a virtual displacement δ^* which will cause additional strain ϵ^* within the element. From (3-9) we get:

$$\epsilon^* = B A^{-1} \delta^* \quad (3-11)$$

The additional strain energy stored by the element during this step in an incremental volume is $\epsilon^{*T} \sigma dv$, which gives for the whole element, by substituting from equations (3-10) and (3-11)

Increase in strain energy

$$\begin{aligned} &= \int_{vol} [B A^{-1} \delta^*]^T [D B A^{-1} \delta - D \epsilon_0] dv \\ &= \int_{vol} [\delta^{*T} [A^{-1}]^T B^T] [D B A^{-1} \delta - D \epsilon_0] dv \end{aligned} \quad (3-12)$$

When the virtual displacement is imposed, the external forces do an amount of work given by

$$\text{External work} = \{\delta^*\}^T \{f\} \quad (3-13)$$

Equating external and internal work done and arranging:

$$\{f\} = [A^{-1}]^T \int_{vol} [B]^T [D] [B] dv [A^{-1}]\{\delta\} - [A^{-1}]^T \int_{vol} [B]^T [D] \{\epsilon_0\} dv \quad (3-14)$$

let

$$[K] = [A^{-1}]^T \int_{vol} [B]^T [D] [B] dv [A^{-1}] \quad (3-15)$$

then

$$\{f\} = [K] \{\delta\} - [A^{-1}]^T \int_{vol} [B]^T [D] \{\epsilon_0\} dv \quad (3-16)$$

Equation (3-16) relates forces to displacement through $[K]$ which is the stiffness matrix¹ determined by (3-15), and if we let:

$$\{f_0\} = [A^{-1}]^T \int_{vol} [B]^T [D] \{\epsilon_0\} dv \quad (3-17)$$

¹ Which will be referred to later in its global banded form as $[S]$.

Then equation (3-16) can be rewritten as

$$\{f\} + \{f_0\} = [K] \{b\} \quad (3-18)$$

In this expression, the components of $\{f\}$ are the actual loads that are imposed on the element and $\{f_0\}$ components are fictitious forces being in fact the nodal forces that would produce the strain $\{\epsilon_0\}$, but this need not be recognized

To solve a problem we merely calculate $\{f_0\}$ for each element, given ϵ_0 , treat it as though it is a real load and add its components to the proper components of the force vector $\{F\}$. This can be conveniently done while the stiffness matrix is being generated. After all displacements for the assembly have been determined, those that apply to each element are selected in turn and the stresses are found by substituting into equation (3-10)

Having developed the theoretical formulation of the stiffness matrix for any finite element, the next step is to choose the element to be employed in this analysis

The cost of a finite element analysis depends on many factors, among which the type of element chosen in the analysis is one factor. The type of element implies the number of elements, the number of degrees of freedom, and the bandwidth to be used

The constant strain triangular element (CST) is a simple element with three corner nodes and with two degrees of freedom at each node. The two degrees of freedom are two displacements perpendicular to each other as shown in Fig. 3-2(a). Because of its simplicity, a CST element does not give good results unless its size is reduced in areas where rapid change of stress is expected. Reducing the size of the element, results in a larger number of data points to be plotted and hence in better curve fitting.

The triangular shape of CST element enables one to break the mesh easily into smaller elements. Moreover, the CST elements do not require a large bandwidth in the stiffness matrix as compared to, for example, linear strain triangular (LST), elements. The usage of LST elements may roughly double the bandwidth which would mean doubling the space required to store the stiffness matrix in the computer memory.

Because of the simple geometry of the problem, the rectangular element would be as good as a triangular element.

The bar element shown in Fig. 3-2(b) has two degrees of freedom at each end. The two degrees of freedom are two displacements perpendicular to each other. The bar element is assumed to model the steel reinforcement and could be used also to model any increase in stiffness (ie lintel). The bar element is chosen because its extremely simple and it can satisfactorily represent the reinforcement steel bar. Moreover, the bar element also guarantees the compatibility of the displacement degrees of freedom at its common points with the CST.

Both the constant strain triangle, and the bar elements are very well investigated in the finite element literature. Hence, these elements have not been developed here. In this analysis, the constant strain triangle is used to model masonry material while the bar element is used to model steel reinforcement.

(a) Constant strain triangle element (CST)

(b) Bar element

Fig 3.2 Elements employed in the analysis

III.5 MASONRY MATERIAL CHARACTERISTICS

Table 3.1 presents the range over which masonry and reinforcing steel characteristics vary. Due to the wide range of variation in the masonry materials, only typical strains and deformational properties will be assumed in this analysis work. The values presented in Table 3.1 are taken from different sources [4], [7], [26], [27], [28] and [31]. In this thesis, Young's modulus, coefficient of expansion, moisture strain and Poisson's ratio values for masonry walls will be taken as, 12000 MPa, 9.5×10^{-6} per $^{\circ}\text{C}$, 0.025% and 0.2 respectively. Similarly, for reinforcement, the assumed properties will be, 200 000 MPa, 12×10^{-6} per $^{\circ}\text{C}$ and 0.3 for Young's modulus, coefficient of thermal expansion, and Poisson's ratio respectively.

Table 3.1 Material Characteristics

Property	Material	Information and Remarks
Shrinkage	Brick masonry	Negligibly small
	Block masonry	-(0.02 to 0.06)% for normal-weight block -(0.02 to 0.07)% for light-weight block
Creep	Brick masonry	(0.4 to 1.4) times elastic strain
	Block masonry	(20 to 25)% of creep in concrete strains are about 3.3 to 6.6 times elastic
Thermal	Brick masonry	$\alpha = 6.12 \text{ to } 6.48 \times 10^{-6} / ^{\circ}\text{C}$
	Block masonry	$\alpha = 9.36 \text{ to } 10 \times 10^{-6} / ^{\circ}\text{C}$
	Reinforcement	$\alpha = 12 \times 10^{-6} / ^{\circ}\text{C}$
Moisture	Brick masonry	0.03 to 0.20 % about 0.025% for non-expansive mortar
	Block masonry	0.0085 to 0.065 %
Young's modulus	Brick masonry	
	and	
	Block masonry	700 to 1000 f'_m or 20 000 MPa maximum
Poisson's ratio	Reinforcement	200 000 MPa
	Brick masonry	0.17 to 0.4
	Block masonry	0.17 to 0.3
	Reinforcement	0.3

III.6 BOUNDARY CONDITIONS

(i) Thermal and Moisture Boundary Conditions

The most direct way of determining the temperatures that actually occur in buildings is to measure them. This approach is quite impractical, however, when the purpose is to find the extreme temperatures that any wall or roof surface may experience, this would require a long series of observations on structures with every combination of wall and roof construction, surface colour, orientation and location.

The daily range of air temperature in the Ottawa area varies from a night temperature of 23°C to a sol air temperature of 59°C in July, for a difference of 37°C . The corresponding range during the winter may be from -30°C to $+24^{\circ}\text{C}$ for a difference of 54°C . According to HUTCHEON, [83], it may be assumed that these are also the approximate ranges of wall surface temperature, and of the mean temperature of the outer brick wythe, for the wall of Fig. 3.3. It is significant that daily temperature variations may be as great as 60 per cent of the maximum annual ranges particularly for situations with low winter temperature [83].

Conditions may differ from those illustrated in Fig. 3.3. Wind reduces surface temperatures under sunny conditions by increasing the heat loss from the surface through convection. Also light-coloured surfaces with relatively low absorptivities for solar radiation will achieve a markedly lower surface temperature. Incident radiation may be increased by reflection and radiation from adjacent surfaces. In particular walls may contribute significantly to the radiation received by adjacent low roofs. There is no reported evidence about how the actual wall surface temperature profile varies from foundation to roof level. Generally, it is believed that the roof temperature is higher in absolute value than that of the foundation (due to heat loss into the ground), but it is hard to say for sure how much this difference could be.

Since tensile stress is of prime interest as far as masonry cracking is concerned, only the case of temperature drop is considered here. The daily temperature fluctuation of 54°C , as shown in Fig. 3.3, has been adopted in this analysis. An average initial wall surface temperature of $+10^{\circ}\text{C}$ was assumed from which a temperature drop to -34°C is applied at the wall foundation level, which results in a total temperature drop of -44°C at the wall bottom edge. In this analysis work, the wall top edge temperature is taken to be 20°C higher or lower (depending on the time of year) than that of the foundation edge of the wall, while linear variation is assumed in between. In other words, the wall is assumed to have a uniform surface temperature of $+10^{\circ}\text{C}$ from which it drops to -34°C at the bottom edge and -54°C at the roof level and varies linearly for intermediate points. This produces the assumed thermal effect, shown in Fig. 3.4(a), of the order of -44°C at the wall bottom and -64°C at the wall top edge. The actual values used are arbitrary and designed to reflect the daily temperature fluctuation. Added to this temperature profile is a -26°C uniform temperature drop, shown in Fig. 3.4(b), to account for an assumed typical shrinkage of 0.025% resulting from moisture effect. This results in the final combined temperature profile shown in Fig. 3.4(c) as -60°C at the foundation edge of the wall and -80°C at the roof edge of

the wall. A linear variation in temperature is assumed between the two limits. The temperature profile shown in Fig. 3.4(c) is assumed to model the daily fluctuation of Ottawa winter temperature plus a typical shrinkage of 0.025°C.

Fig. 3.3 Daily and annual range of temperatures for an insulated masonry wall at latitude 45° N, facing south (temperature gradients calculated from steady state theory). The given values are for Ottawa conditions, Ref. [83].

Fig 3.4 Assumed thermal and equivalent moisture profiles.

(ii) Geometry and Restraint Conditions

The analysis has been carried out on fairly substantial walls having lengths of 50, 25, 12.5 and 6.25 m. The height as well as the thickness was taken to be constant throughout the analysis with values of 5 m and 250 mm respectively. Hence the walls considered have aspect ratios, height to length, of 0.1, 0.2, 0.4 and 0.8. The dimensions were chosen to cover a wide range of low-rise commercial masonry buildings. In all of the cases considered, no slip or relative movement was allowed at the wall-foundation interface.

Table 3.2 shows the different displacement and boundary conditions used to simulate the situation in practice. The seven cases that have been considered are presented in the following order:

Case 1

In this case, four solid walls fixed at the bottom were considered, having aspect ratios of 0.1, 0.2, 0.4 and 0.8 respectively. In order to study the effect of changing the aspect ratio on the thermal and moisture stresses, the walls were subjected to the linear temperature profile shown in Fig. 3.4(c).

Case 2

Four walls of aspect ratios 0.1, 0.2, 0.4 and 0.8 were considered in this case. A door opening of dimensions $3 \times 3 \text{ m}^2$ is introduced in each of the four walls in order to study the effect of the presence of a door opening on thermal and moisture induced stresses. The walls were taken to be fixed at the bottom and were subjected to a linear temperature profile.

Case 3

Three walls of aspect ratios 0.1, 0.2 and 0.4 were considered in this case. A door and two window openings having dimensions of $3 \times 3 \text{ m}^2$ and $2 \times 1 \text{ m}^2$ respectively were introduced in each of the three walls. In order to study the effect of the presence of more than one opening on thermal and moisture stresses, the walls were subjected to a linear temperature profile. The walls were taken to be fixed at the bottom. The 0.8 aspect ratio was omitted since its length can not accommodate the two openings.

Case 4

Four walls of aspect ratios 0.1, 0.2, 0.4 and 0.8 were considered in this case. The walls were taken to be fixed at top and bottom in order to study the effect of top edge restraint on thermal and moisture stresses. Again a linear temperature profile was applied.

Case 5

For non-loadbearing walls in seismic zone 2 (Ottawa area), the 1978 CSA code [27] requires that a minimum reinforcement area of $0.0005 A_g$ be provided, where A_g is the gross cross-sectional area of the wall. Four walls of aspect ratios 0.1, 0.2, 0.4 and 0.8 are considered in this case. The walls were reinforced horizontally with 1 M15 bar at 1.2 m c/c to investigate the effect of minimum reinforcement on thermal and moisture stresses. The walls were subjected to a linear temperature profile with the shrinkage equivalent temperature applied to masonry elements only.

Case 6

In this case, the linear temperature profile is compared with a stepped temperature profile. The four walls of Case 1 were analyzed again under a uniform temperature drop of -60°C for the wall and -80°C at the roof level. The roof was represented by the upper 30 cm part of the wall. The modulus of elasticity was taken to be 20 000 MPa for the upper wall elements.

Case 7

In this case, the effect of loadbearing type of construction was combined with thermal and moisture effects.

Table 4.1.6.A of the 1980 National Building Code of Canada, [84], gives 12 kN/m^2 as the minimum specified live load for trucking spaces. An extra live load of 2 kN/m^2 to account for roof self weight, mechanical and electrical equipment was assumed. Then for a typical exterior loadbearing masonry wall, carrying a 5 m width roof strip of a one way span system:

$$\text{total load} = 5 \times 14 = 70\text{ kN/m}$$

This load was taken to be uniformly distributed along the top of the wall in this case. The three aspect ratios considered were 0.1, 0.2 and 0.4. The aspect ratio of 0.8 was considered to be usually a part of a two way spanning system and hence it has been omitted. A linear temperature profile was imposed.

Table 3.2 Geometry and Restraint Conditions.

Case	Aspect Ratio	Information and Remarks
1	0.1	Solid wall fixed at the bottom, linear temperature profile
	0.2	
	0.4	
	0.8	
2	0.1	Symmetrically located $3 \times 3 \text{ m}^2$ door opening, wall fixed at the bottom, linear temperature profile
	0.2	
	0.4	
	0.8	
3	0.1	$3 \times 3 \text{ m}^2$ door plus $2(2 \times 1 \text{ m}^2)$ windows, symmetrically located, wall fixed at the bottom, linear temperature profile
	0.2	
	0.4	
	0.8	
4	0.1	Solid wall, top and bottom, fixed, linear temperature profile
	0.2	
	0.4	
	0.8	
5	0.1	Solid wall reinforced with 1 M15 bar at 1.2 m c/c, fixed at the bottom, linear temperature profile
	0.2	
	0.4	
	0.8	

Table 3.2 Continued

Case	Aspect Ratio	Information and Remarks
6	0.1	Solid wall,
	0.2	fixed at the bottom,
	0.4	stepped
	0.8	temperature profile
7	0.1	Solid wall fixed at
	0.2	the bottom carrying,
	0.4	70 kN/m uniformly distributed
		load at top, linear temperature profile

III.7 COMPUTER PROGRAM TO SOLVE FOR THERMAL STRAINS

The work presented in section III 4 was incorporated in a structural finite element analysis program. Most of the subroutines used were taken from BOWES and RUSSELL [82]

As shown in the flowchart of Table 3 3, the program follows the steps mentioned in section III 4 by finding the stiffness of each element for both the triangular elements (wall elements) and the bar elements (reinforcement elements) and adds the elements stiffness matrix to the global stiffness matrix $[S]$ having dimensions of band width limit or maximum band width times number of degrees of freedom

Similarly the program builds the force vector $\{F\}$ by adding the thermal force vector for each element of both materials (masonry and reinforcement) to the global force vector $\{F\}$. The program also takes the masonry self-weight into consideration.

When all stiffnesses and forces corresponding to each element have been incorporated into $[S]$ and $\{F\}$ respectively, the program then solves for $\{U\}$, where $\{U\}$ is the displacement vector, which, like vector $\{F\}$, has some of its components known while others are unknown. The program first uses the Payne and Irons technique to separate the knowns from the unknowns in the stiffness equation $\{F\} = [K]\{U\}$ and then solves for $\{U\}$ by back substitution and Gauss elimination. Knowing $\{U\}$, the program proceeds to equation (3-18) and calculates the $\{F\}$ vector. At this stage all the nodal forces and displacements are known. The program then goes back to each wall element and by using equation (3-10) the program calculates and prints horizontal stresses S_{xx} , vertical stresses S_{yy} and shear stresses S_{xy} . The program as well calculates and prints the major and minor principal stresses P_1, P_2 respectively and the angle of inclination of the plane on which the major and minor stresses occur, θ . Similarly the program proceeds to the reinforcement elements and calculates their axial stresses and prints them out. Having finished one case the program either terminates the execution, which is done by reading NEXT=0, or proceeds to another case by reading NEXT=1 in the data file. The sign convention followed in this program is the usual convention, identical to the plane coordinates sign for forces and displacements while that of stress is positive for tension and negative for compression. Since the program is designed to solve for in-plane forces and thermal strains, any deformations other than thermal have to be converted to an equivalent rise or fall in temperature in the input data.

Table 3.3 Flowchart of the Computer Program

```

1      Print Title
      Read and Print Case Title
      Read Nodes Coordinates
      Read and Write Wall Elements Data
      Read and Write Reinforcement Properties
      Read Reinforcement Elements Data
      Fill in [S], {U} and {F} With Zeros
      Fill in [D] and [B] Matrices
      Fill in  $\bar{A}$  Matrix
      Find  $A^{-1}$  and Write on 1
      Number of Wall Elements = 0 GO TO 22
      Calculate Wall Element Stiffness Matrix [E] and Thermal
      Force Vector FTE and add to [S] and {F}
      Calculate Work Equivalent Vector for Elements Own Weight
      and Add to {F}
22     Number of Reinforcement Elements = 0 GO TO 77
      Find Reinforcement Element Stiffness and add to [S]
      Find Reinforcement Elements Thermal Force Vector and add to {F}
77     Write [S] on 4
      Read Known Forces and add to {F}
      Read Known Displacements and add to {U}
      Solve for {U} by method of Payne and Irons, Gauss elimination
      Read [S] from 4
      Solve {F} = [S].{U} Print F and U
      Read  $A^{-1}$  from 1
      For All Wall Elements Find and Print the 3 Plane Stresses
      Major and Minor Principal Stresses and Their Directions
      Number of Reinforcement Elements = 0 GO TO 88
      For All Reinforcement Elements Find and Print Axial Stresses
88     Read      N E X T

      N E X T:0

```

< = >

Call EXIT GO TO 1

CHAPTER IV

ANALYSIS RESULTS AND DISCUSSION

IV.1 GENERAL

In this chapter the results of the finite element analysis of thermal and moisture induced stresses in low-rise masonry walls is presented. For ease of discussion the effect of the different factors, mentioned in Table 3.2, on the stress profile will be considered separately. Frequent reference will be made to the cases of Table 3.2, and the presentation will follow the same order. The analysis was done on Carleton University's CP6 computer system (Honeywell). In order to minimize the numerical round off errors, the analysis was carried out in double precision. Due to symmetry, only half of each wall was considered in the analysis and the resultant stresses are also only presented for each half wall.

IV.2 EFFECT OF ASPECT RATIO :Case 1

IV.2.1 Horizontal Stresses

Figures 4.1(a), and (b) illustrate the horizontal stress distributions for the 4 aspect ratios considered for the bottom and top edges of the wall respectively. For all aspect ratios the horizontal stress at the bottom edge starts at a high value of 12.5 MPa at the wall corner, then suddenly drops to a constant value of 8 MPa for 0.1 and 0.2 aspect ratios, and to 7 MPa for 0.4 and 0.8 aspect ratios. Along the top edge, the magnitude of the horizontal stress is maximum at the centerline, as expected, with values of 10, 8, 2, and -1 MPa for 0.1, 0.2, 0.4, and 0.8 aspect ratios respectively. The stress was shown to be zero at the top corner for all aspect ratios. Figures 4.1(a) and (b) indicate that stresses are generally the same for all aspect ratios along the restrained edge of the wall, but along the top edge of the wall the stresses were found to differ substantially. For the same temperature conditions, the longest wall showed the highest tensile stress at the top edge, while, the shortest wall (higher aspect ratio) went into compression at the top edge. It can be concluded that the shorter the wall length, the lesser the tensile thermal stress. It was also seen that doubling the wall length from 12.5 m to 25 m increased the maximum stress at the top edge by 4 times, compared to 1.25 times for an increase from 25 to 50 m. Generally, 0.1 and 0.2 aspect ratio walls are similar with respect to horizontal stress.

IV.2.2 Vertical stresses

Fig. 4.2 illustrates the vertical stress distribution along the bottom edge for Case 1. The vertical stress distribution was found to have a similar pattern for all aspect ratios. The maximum values at the lower corner were found to be 23 MPa in tension for 0.1 and 0.2 aspect ratios and 17 MPa for 0.4 and 0.8 aspect ratios. The stress drops from these maximum values to -2.5 MPa

compressive stress at the centerline for 0.4 and 0.8 aspect ratios, and goes to zero for 0.1 and 0.2 aspect ratios

IV.2.3 Shear Stresses

Shear stress distribution along the bottom edge for Case 1 is illustrated in Fig. 4.3. Starting from relatively high values of 11 to 13 MPa at the bottom corner it drops to zero at the wall centerline. The corner value was the same for all aspect ratios as was the stress pattern.

IV.2.4 Principal Stresses

The principal stress contour lines for the 4 different aspect ratios of Case 1 are shown in Figures 4.4(a), (b), (c), and (d). Along the fixed bottom edge, the value and pattern of the principal stress was shown to be similar for all aspect ratios. Along the free vertical edge the principal stress reduces to zero. The principal stress was high enough to crack the wall in tension near the restrained edge in all the cases, and at the centerline near the top edge in most of the cases. The probable direction of the principal stresses is discussed in section IV.8 of this chapter.

IV.3 EFFECT OF OPENINGS

IV.3.1 Horizontal Stresses

For walls with one opening (ie Case 2), the horizontal stress distributions at the bottom and top edge are shown in Figures 4.5(a) and (b) respectively. For walls with more than one opening (ie Case 3) the horizontal stress distributions are shown in Figures 4.9(a), and (b) for the bottom and top edges respectively. In Case 2, bottom horizontal stress was shown to be 8 MPa for 0.1 and 0.2 aspect ratios at the bottom corner, dropping to 6 MPa at the door opening. For 0.4 and 0.8 aspect ratios, the stress at the base is 7 and 6 MPa respectively at the bottom corner from which it varies in a zigzag pattern till it returns to 6 MPa at the bottom corner of the door opening. For Case 2, the maximum top edge horizontal stress at the wall centerline was shown to have substantially different values for different aspect ratios, while it smoothly drops to zero at the top corner for all aspect ratios. The maximum stress at the top edge was shown to be 12, 9, 2 and -2 MPa, for 0.1, 0.2, 0.4 and 0.8 aspect ratios respectively. *Relating these values to the corresponding stresses of Case 1, one can see that the introduction of one opening has only a small effect on the horizontal stress at the top edge.* For Case 3, top and bottom edge horizontal stresses fall under the same discussion as Case 2, with nearly the same distributions observed in Figures 4.9 and 4.5. *Generally, the introduction of openings of the geometry and dimensions as those of Cases 2 and 3 have little effect on horizontal stress at the top and bottom edges.* The horizontal stress at the edges of the openings is expected to be quite a bit higher however. This is discussed in detail in Section IV.3.4.

IV 3 2 Vertical Stresses

The vertical stress distributions along the bottom edge for Cases 2 and 3 are shown in Figures 4 6 and 4 10 respectively. For Case 2 the values were shown to start in the corner at a stress of 16 MPa for 0 1, 0 2 and 0 4 aspect ratios and at 4 MPa for 0 8 aspect ratio. These stresses then drop to a slight amount of compression before they go again into tension of 5 MPa for 0 1 and 0 2 aspect ratios at the wall centerline while they remain in compression for 0 4 and 0 8 aspect ratios. For Case 3, the vertical stress is shown to have started at high value of 22 MPa for the 3 aspect ratios of 0 1, 0 2 and 0 4. The stress then suddenly drops to slight compression from which it goes back to a tensile stress of about 5 MPa for the 3 aspect ratios. The vertical compression stress near the bottom corner was slightly less in Case 3 than in Case 2. *It can be concluded that more openings in the wall will introduce larger vertical tensile stresses at the base.* Comparing the vertical stresses of Case 2 and 3 with those of Case 1, it can be seen that openings in the wall result in larger vertical tensile stresses at the base of the wall near the openings (5 vs zero MPa). This of course, supports the idea that if a section is weakened by an opening, it will tend to attract more tension.

IV 3 3 Shear Stresses

Figures 4 7 and 4 11 show the shear stress distributions along the bottom edges of Case 2 and 3 respectively. The stress starts at the bottom corner at relatively high values of 14 MPa in Case 3 vs 10 MPa in Case 2. From these high values the stress in both cases drops smoothly to -7 MPa for all aspect ratios in the two cases. Comparing the bottom edge shear stress of Case 1 (Fig 4 3) with that of Case 2 and 3 one can see that *the effect of the presence of an opening results in larger shear stresses near the opening*.

IV 3 4 Principal Stresses

Figures 4 8(a), (b), (c), (d) and 4 12(a), (b), (c) and (d) show the principal stress contours for Cases 2 and 3 respectively. The presence of the door opening in Case 2 led to high principal stresses around the opening corners as would be expected. The same effect was noticed due to the presence of windows. Away from the openings the principal stress distribution was similar to that of Case 1. *It appears likely that cracking of the masonry will occur near the bottom corner of the wall and at the corner of openings.* The principal stress induced around the openings was slightly higher for lower aspect ratios (longer walls).

IV.4 EFFECT OF TOP RESTRAINT

IV.4.1 Horizontal Stresses

Horizontal stresses along the restrained bottom and top edges of Case 4 are shown in Figures 4.13(a) and (b) respectively. The values are seen to be higher at the top than at the bottom because of the higher temperature differential assumed at the top. Both the top and the bottom maximum stresses were higher for smaller aspect ratio (longer) walls. The top restraint in Case 4 resulted in top and bottom stresses greater than those of Case 1 (by 2 MPa). Generally, the presence of top edge restraint resulted in an increase in horizontal stress and hence the possibility of cracking increased. It should be noted again, however, that the top edge has been assumed fully restrained in this case. In real situations the horizontal stress at the top edge will probably not be as high as observed here but will be higher than those predicted by Cases 1, 2, or 3.

IV.4.2 Vertical Stresses

As shown in Fig. 4.14, vertical stress along the bottom edge of Case 4 starts at a value of 25 MPa at the corner, dropping suddenly to a constant value of 10 MPa for 0.1, 0.2 and 0.4 aspect ratios, and 7 MPa for 0.8 aspect ratio. This constant value was substantially higher than that of Case 1, which was less than zero.

IV.4.3 Shear Stresses

The shear stress distribution for Case 4 along the bottom edge is shown in Fig. 4.15. High stress values were recorded at the corner, dropping to zero at the wall centerline following a similar pattern for all aspect ratios. The values and patterns were found to be identical to those of Case 1 and did not seem to be affected by restraining the top edge.

IV.4.4 Principal Stresses

The principal stress contour lines for Case 4 are illustrated in Figures 4.16(a), (b), (c) and (d). It is quite clear that the top restraint has a significant influence on the magnitude of the principal stress. The principal stresses at the top of the 0.8 aspect ratio wall, for instance, increased from zero in Case 1, to as high as 16 MPa in Case 4. A similar observation holds true for the other aspect ratios.

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IV.5 EFFECT OF REINFORCEMENT

IV.5.1 Horizontal Stresses

Horizontal stress distributions at bottom and top edges for Case 5 are illustrated in Figures 4.17(a) and (b) respectively. The stress was shown to have a high value of 12.5 MPa at the lower corner of wall and it abruptly decreased to a constant value of 7.5 MPa for all aspect ratios. On the top face of Case 5 the horizontal stress is shown to be zero at the top corner of the wall, increasing in a bell shaped pattern, to its maximum value at the wall centerline. The maximum top edge horizontal stress was shown to be higher for smaller aspect ratios, with values of 10, 8, 2 and zero MPa for 0.1, 0.2, 0.4 and 0.8 aspect ratios respectively. On comparison with the corresponding stress of Case 1, *it was noted that stresses in both cases were identical in value and pattern.* Hence, introducing reinforcement of the amount and layout shown in Case 5, had no effect on the horizontal stresses.

IV.5.2 Vertical Stresses

Vertical stress distribution, at the bottom edge, for Case 5 is illustrated in Fig. 4.18. The stress was shown to have a maximum value of 23 MPa at the bottom corner reducing abruptly to zero for 0.8 aspect ratio wall and a compressive stress of -2 MPa for other aspect ratios. These stresses were observed to be identical to those of Case 1. *It could be concluded that reinforcing Case 1 walls, with the amount shown in Case 5, has no effect on the vertical stress.*

IV.5.3 Shear Stresses

Lower edge shear stresses for Case 5 are illustrated in Fig. 4.19. Shear stress values and patterns were found to be similar to the corresponding ones for Case 1, which indicates that introduction of reinforcement, such as in Case 5, has no effect on the shear stress for all of the aspect ratios.

IV.5.4 Principal Stresses

As in Figures 4.20(a), (b), (c) and (d), the principal stress contour lines for Case 5 are shown to be identical to those of Case 1. From these similarities, one can conclude that the principal stresses are not altered by introducing 15M reinforcement bar in the walls of Case 1, laid out as in Case 5.

The similarity between the recorded stresses in Cases 1 and 5 clearly showed that the minimum reinforcement required by the CSA code has a negligible effect on the stress values and patterns. This similarity is due to two reasons; firstly the coefficient of thermal expansion of reinforcement is of the same order of magnitude as that of masonry, and secondly, the area of such an amount of reinforcement is negligibly small compared to the wall cross-sectional area.

IV.6 EFFECT OF TEMPERATURE PROFILE

IV.6.1 Horizontal Stresses

Bottom and top edge horizontal stress distributions for the stepped temperature profile of Case 6 are illustrated in Figures 4.21(a) and (b). The stresses are similar to those of Case 1 except for a slight increase of the maximum value observed at the top edge in the 0.1 aspect ratio wall (from 10 MPa in Case 1, to 12 MPa in Case 6). This shows that using the same temperature drop in a stepped rather than linear profile did not have a significant effect on horizontal stresses.

IV.6.2 Vertical Stresses

As shown in Fig. 4.22, the vertical stress distribution for Case 6 is the same as that of Case 1. It starts from its highest value at the bottom corner, abruptly decreasing to a small compressive stress at a point near the corner, then gradually returns to zero in the longer walls. The maximum corner stress was found to be of 22 MPa for 0.1 and 16 MPa for 0.8 aspect ratios. Generally, changing the temperature profile from linear to stepped did not have any effect on the vertical stresses.

IV.6.3 Shear Stresses

Fig. 4.23 illustrates the shear stress distribution along the bottom edge of Case 6 walls. The shear stress was observed to be identical to that of Case 1. It starts from its highest value at the bottom corner and drops smoothly to zero at the wall centerline. The use of a stepped temperature profile does not change the shear stress as long as the same temperature range is maintained.

IV.6.4 Principal Stresses

The principal stress contour lines for Case 6 are shown in Figures 4.24(a), (b), (c) and (d). These curves show that the stepped temperature profile results in an increase in principal stresses in the upper regions of the walls over that of the linear profile. For example, for the 0.1 aspect ratio wall near the centerline, the stress increased from 8 MPa (Case 1) to 13 MPa (Case 6). The stress was nearly the same for both the linear and stepped profiles for 0.8 aspect ratio, starting at 12 MPa near the bottom corner and dropping to zero MPa in the upper third of the wall.

IV.7 EFFECT OF LOADBEARING

IV.7.1 Horizontal Stresses

Figures 4.25(a) and (b) show the bottom and top edge horizontal stress distributions for the loadbearing masonry wall of Case 7. On the bottom edge of the wall the stress starts at a value of 12.5 MPa, dropping to a constant value of 7.5 MPa, for 0.1 and 0.2 aspect ratios and to 7 MPa for 0.4 aspect ratio. At the top edge, the horizontal stress reached maximum values of 10, 7.5 and 2 MPa at the wall centerline for 0.1, 0.2 and 0.4 aspect ratios respectively. The top and bottom horizontal stresses were observed to be identical to the corresponding stresses of Case 1.

IV.7.2 Vertical Stresses

As illustrated in Fig. 4.26, the vertical stress at the bottom edge of Case 7 is similar to the corresponding stress of Case 1, which was expected since the stress level induced by loadbearing effect is negligibly small (about .28 MPa). It starts from a maximum value of 24 MPa at the corner of the wall and reduces to a compressive stress of 3 MPa at a distance of 2 m from the vertical edge. From this compression value the stress then tends to zero at the wall centerline for 0.1 and 0.2 aspect ratios.

IV.7.3 Shear Stresses

Fig. 4.27 shows the shear stress distribution at the bottom edge of Case 7. The stress distributions is identical to that observed in Case 1, starting at 12 MPa in the corner and reducing to zero at the centerline.

IV.7.4 Principal Stresses

The principal stress contour lines of Case 7 are shown in Figures 4.28(a), (b) and (c). These stresses were found to be the same as those observed in Case 1, starting from a relatively high value of 14 MPa near the lower corner, reducing to a moderate value of 7 to 9 MPa near the wall centerline and to zero near the upper corner.

IV.8 DISCUSSION

Several walls have been analyzed here for the range of temperatures discussed in chapter III, combined with the effect of openings and restraint conditions also discussed in chapter III.

For a typical ultimate compressive strength of masonry $f'_m = 12$ MPa, Table 5 of the 1978 CSA code [27], gives:

Maximum allowable compressive stress	$= 0.225 f'_m$
	$= 2.7$ MPa
Maximum allowable flexural tensile stress (parallel to bed joint)	$= 0.32$ MPa
Maximum allowable flexural tensile stress (normal to bed joint)	$= 0.16$ MPa
Maximum allowable shear stress	$= 0.23$ MPa
Maximum allowable bearing stress	$= 0.25 f'_m$
	$= 3$ MPa

From the results presented in the previous sections, it can be observed that the predicted stresses are much higher than those allowed by the CSA code. In most of the cases horizontal stress at the bottom corner of the wall and at the top edge centerline were well beyond the code allowable values. Hence under the thermal and moisture conditions assumed in this analysis, the walls would almost certainly have shown some cracking. These cracks most likely would start at the lower corner, at the corners of openings and at the centerline near the top edge. The cracks would probably be wider and easily visible in smaller aspect ratio (longer) walls.

Horizontal stress at the lower corner (ie Fig. 4.1(a)) did not vanish as theory demands due to singularity of the finite element solution at this point.

As shown in the Appendix, the meshes were refined at the fixed bottom edge to improve the accuracy of the analysis. This refinement led to the edge elements to have higher stresses than those away from the lower edge thus causing the zigzag pattern in some of the lower edge graphs.

Vertical stresses were also highest at the lower corner, and the calculated values were easily large enough to crack the wall near the lower corner. The vertical stresses at the lower corners of the openings were also high enough to result in cracking.

Shear stresses at the lower face were higher than those permitted by the CSA code, particularly at the corners. This would likely result in horizontal cracks at the base of the wall as the wall slides over the foundation.

The principal stress contour lines show quite clearly the most likely locations of crack initiation (at wall and opening corners) and their probable orientation (parallel to the contour lines).

The stepped profile temperature drop resulted in somewhat higher principal stresses near the roof than that predicted by the linear profile. Since field observations (Chapter II) indicate that the roof-wall interface is a common cracking problem area, the stepped profile may be a better model to employ in further cracking analysis work.

The principal stresses near the roof are dramatically increased if the roof is considered a restraining boundary (as in Case 4). Under these conditions, cracking is most likely to occur first

at corners of walls and openings with shallow or horizontal orientations. Cracks which occur away from the edges are likely to be vertical.

The principal function of the reinforcement is to resist tensile forces after masonry cracking and hence to control crack widths. Minimum amounts of reinforcement specified in the 1978 CSA code [27] generally is thought to ensure the presence of many fine cracks instead of fewer larger ones. To what degree this minimum amount of steel has been effective in controlling cracks from thermal and moisture movements in actual structures is not known and has never been investigated. The use of reinforcement, however, involves the risk of corrosion once this reinforcement comes in contact with moisture; such corrosion will reduce the effect of the reinforcement and may result in spalling of masonry. Shrinkage of masonry will lead to stresses being set up in the reinforcement. The stress level in the reinforcement was moderate (about 100 MPa) in compression*. The presence of such steel stresses likely will contribute to masonry cracking.

The loadbearing masonry type of construction showed nearly similar stresses to the nonload-bearing one because the stress levels induced by the loadbearing effect in low-rise masonry walls are very small.

The horizontal stress distribution along the centerline for Case 1 is shown in the Appendix (Fig. A-11), and was found to have close pattern to the stress distribution for similar walls reported by STOFFERS, [87].

* The stress level in the reinforcement was moderate (from about 100 MPa in compression for 0.8 aspect ratio to 57 MPa in tension near wall mid-length for 0.1 aspect ratio), this stress corresponding to the assumed 0.025% typical masonry shrinkage.

Fig. 4.1(a) Horizontal stress distribution, bottom edge, Case 1:solid wall, linear temperature profile.

Fig. 4.1(b) Horizontal stress distribution, top edge, Case 1: solid wall, linear temperature profile.

Fig. 4.2 Vertical stress distribution, bottom edge Case 1: solid wall, linear temperature profile.

(a) For 0.8 aspect ratio.

(b) For 0.4 aspect ratio.

(c) For 0.2 aspect ratio.

(d) For 0.1 aspect ratio.

Fig. 4.4 Principal stress contour lines, Case 1: solid wall, linear temperature profile, numbers represent tensile stress in MPa.

Fig. 4.5(a) Horizontal stress distribution, bottom edge, Case 2: wall with door, linear temperature profile.

Fig. 4.5(a) Horizontal stress distribution, bottom edge, Case 2: wall with door, linear temperature profile.

Fig. 4.5(b) Horizontal stress distribution, top edge, Case 2: wall with door, linear temperature profile.

Fig. 4.6 Vertical stress distribution, bottom edge, Case 2: wall with door, linear temperature profile.

Fig. 4.7 Shear stress distribution, bottom edge, Case 2: wall with door, linear temperature profile.

(a) For 0.8 aspect ratio.

(b) For 0.4 aspect ratio.

(c) For 0.2 aspect ratio.

(d) For 0.1 aspect ratio.

Fig. 4.8 Principal stress contour lines, Case 2: wall with door, linear temperature profile, numbers represent tensile stress in MPa.

Fig. 4.9(a) Horizontal stress distribution, bottom edge, Case 3: wall with door and 2 windows, linear temperature profile.

Fig. 4.9(b) Horizontal stress distribution, top edge, Case 3: wall with door and 2 windows, linear temperature profile.

Fig. 4.10 Vertical stress distribution, bottom edge, Case 3: wall with door and 2 windows, linear temperature profile.

Fig. 4.11 Shear stress distribution, bottom edge, Case 2: wall with door and 2 windows, linear temperature profile.

(a) For 0.4 aspect ratio.

(b) For 0.2 aspect ratio.

(c) For 0.1 aspect ratio.

Fig. 4.12 Principal stress contour lines, Case 3: wall with door and 2 windows
linear temperature profile, numbers
represent tensile stress in MPa.

Fig. 4.13(a) Horizontal stress distribution, bottom edge, Case 4: solid wall fixed at top and bottom, linear temperature profile.

Fig. 4.13(b) Horizontal stress distribution, top edge, Case 4: solid wall fixed at top and bottom, linear temperature profile.

Fig. 4.14 Vertical stress distribution, bottom edge, Case 4: solid wall fixed at top and bottom, linear temperature profile.

Fig. 4.15 Shear stress distribution, bottom edge, Case 4: solid wall fixed at top and bottom, linear temperature profile.

(a) For 0.8 aspect ratio.

(b) For 0.4 aspect ratio.

(c) For 0.2 aspect ratio.

(d) For 0.1 aspect ratio.

Fig. 4.16 Principal stress contour lines, Case 4: solid wall fixed at top and bottom, linear temperature profile, numbers represent tensile stress in MPa.

Fig. 4.17(a) Horizontal stress distribution, bottom edge, Case 5: solid wall horizontally reinforced with 1 M15 bar at 1.2 m c/c, linear temperature profile.

Fig. 4.17(b) Horizontal stress distribution, top edge, Case 5: solid wall horizontally, reinforced with 1 M15 bar at 1.2 m c/c, linear temperature profile.

Fig. 4.18 Vertical stress distribution, bottom edge, Case 5: solid wall horizontally reinforced with 1 M15 bar at 1.2 m c/c, linear temperature profile.

Fig. 4.19 Shear stress distribution, bottom edge, Case 5: solid wall horizontally reinforced with 1 M15 bar at 1.2 m c/c, linear temperature profile.

(a) For 0.8 aspect ratio.

(b) For 0.4 aspect ratio.

(c) For 0.2 aspect ratio.

(d) For 0.1 aspect ratio.

Fig. 4.20 Principal stress contour lines, Case 5: solid wall horizontally reinforced with 1 M15 bar at 1.2 m c/c, linear temperature profile, numbers represent tensile stress in MPa.

Fig. 4.21(a) Horizontal stress distribution, bottom edge, Case 6: solid wall, stepped temperature profile.

Fig. 4.21(b) Horizontal stress distribution, top edge, Case 6: solid wall, stepped temperature profile.

Fig. 4.22 Vertical stress distribution, bottom edge, Case 6: solid wall, stepped temperature profile.

Fig. 4.23 Shear stress distribution, bottom edge, Case 6: solid wall, stepped temperature profile.

Fig. 4.24 Principal stress contour lines, Case 6: solid wall, stepped temperature profile, numbers represent tensile stress in MPa.

Fig. 4.25(a) Horizontal stress distribution, bottom edge, Case 7: solid wall loaded with 70 kN/m uniformly distributed on top, linear temperature profile.

Fig. 4.25(b) Horizontal stress distribution, top edge, Case 7: solid wall loaded with 70 kN/m uniformly distributed on top, linear temperature profile.

Fig. 4.26 Vertical stress distribution, bottom edge, Case 7: solid wall loaded with 70 kN/m uniformly distributed on top, linear temperature profile.

Fig. 4.27 Shear stress distribution, bottom edge, Case 7: solid wall loaded with 70 kN/m uniformly distributed on top, linear temperature profile.

(a) For 0.4 aspect ratio.

(b) For 0.2 aspect ratio.

(c) For 0.1 aspect ratio.

Fig. 4.28 Principal stress contour lines, Case 7: solid wall loaded with 70 kN/m uniformly distributed on top, linear temperature profile, numbers represent tensile stress in MPa

CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS

V.1 CONCLUSIONS

An investigation was carried out on masonry walls to study the effect of moisture and temperature change on the stress distribution in walls. In the analysis, seven cases with aspect ratios ranging from 0.1 to 0.8 have been considered.

A plane stress analysis was performed using finite element technique. The masonry was assumed to be a homogeneous, linearly elastic material. Conclusions derived from the analysis results are presented in point form.

1. An increase in wall length increases thermal stresses and hence the possibility of cracking. However, this increase is not significant for aspect ratios (H/L) less than about 0.2.
2. The presence of openings results in stress concentrations at opening edges (particularly corners). Hence, there is a greater probability of cracking around openings.
3. Restraining the top edge results in higher tensile stresses being induced in the wall regions close to the restrained region. Highly restrained walls are more vulnerable to cracks due to moisture and thermal effects. Whenever possible, rigid connections between the roof and the wall should be avoided.
4. Because reinforcement has only a slightly higher coefficient of thermal expansion than masonry, it will not affect the stresses arising in the masonry due to thermal effects to any significant degree. However it could be expected to provide considerable local restraint in the event of a crack, thus limiting the crack width.
5. The stepped temperature profile results in higher stresses near the roof as compared to the linear profile. As a model of a semi-rigid roof undergoing a larger temperature drop than that of the wall to which the roof is attached, it gives results that agree quite well with field observations.
6. Under the same temperature profile, loadbearing and nonloadbearing walls produce similar stress distributions.
7. Generally, the temperature fluctuation and the degree of restraint are the two most important factors in the magnitude of stress developed in low-rise masonry walls.
8. In practice, poor workmanship, inadequate design and detailing or poor materials may result in cracking under even minor thermal gradients and shrinkage effects.
9. The designer has little or no control over temperature fluctuations and so must rely on controlling the restraint conditions to avoid cracking. This analysis indicates that the provision of vertical control joints, even at a spacing of $H/L = 0.8$, is insufficient on its own to eliminate cracking. It is probably much more economical, however, to provide for the occurrence of

cracking in the design by limiting individual crack widths through the use of reinforcement and the provision of vertical control joints.

10. The fact that recently built masonry structures still suffer from cracking problems even though the reasons behind such cracks are reasonably well known indicates that designers lack either the information about how such cracks can be prevented or they may not be fully aware of the seriousness of the cracking problems. This lack of understanding necessitates that better ways of communication be developed between designers and institutions involved in masonry research and education.
11. Part of the thermal and moisture deformations can be eliminated by careful choice of materials, construction and design. Factors such as proper inspection, weather protection of materials and of newly constructed masonry, as well as the minimizing of stiff connections will all help alleviate potential cracking problems.

V.2 RECOMMENDATIONS FOR FUTURE RESEARCH

In order to achieve both a better understanding of the long term masonry deformations and reliable provisions for spacing and frequency of control joints, there is an urgent need for further investigation on this subject. More field observations, laboratory testing, numerical analyses and theoretical studies are of both academic and practical interest.

Suggested research topics may include:

- i- An extensive probabilistic analysis to be carried out that can lead to a rational combination of the elements involved in the cracking problem. The probability of occurrence of the factors that contribute to the problem of cracking is still based on judgement rather than analytical or experimental evidence.
- ii- Non-linear (elastic or plastic) numerical analyses using finite element method should be carried out. In such analysis work a reliable failure criteria should be employed; the location of the first crack and how it propagates, as well as reinforcement and control joint requirements to limit distress, should be the goals of such a study.
- iii- Long term field measurements should be carried out parallel to and in conjunction with the suggested research topics. A survey of this type should last over a long enough time period to allow major deformations to take place.
- iv- Large and model scale laboratory testing programs with walls under different controlled environments should be investigated for separate and/or combined deformations. The outcome of such testing programs can be used to prove, disprove or adjust any analytical results.
- v- Development of a theoretical or mathematical model that is capable of representing the various factors involved in the cracking process. The analysis results of such a model can also serve as a basis for numerical, laboratory or field studies.

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APPENDIX

Fig. A-1 Finite element mesh for 0.8 aspect ratio solid wall.

Fig. A-2 Finite element mesh for 0.4 aspect ratio solid wall.

Fig. A-3 Finite element mesh for 0.2 and 0.1 aspect ratio solid wall.

Fig. A-4 Finite element mesh, for 0.8 aspect ratio wall with $3 \times 3 \text{ m}^2$ door opening.

Fig. A-5 Finite element mesh, for 0.4 aspect ratio wall with $3 \times 3 \text{ m}^2$ door opening.

Fig. A-6 Finite element mesh for 0.2 aspect ratio wall with $3 \times 3 \text{ m}^2$ door opening.

Fig. A-7 Finite element mesh for 0.1 aspect ratio wall
with $3 \times 3 \text{ m}^2$ door opening.

Fig. A-8 Finite element mesh for 0.4 aspect ratio wall with $3 \times 3 \text{ m}^2$ door plus $2(2 \times 1 \text{ m}^2 \text{ window})$ opening

Fig. A-9 Finite element mesh for 0.2 aspect ratio wall with $3 \times 3 \text{ m}^2$ door plus $2(2 \times 1 \text{ m}^2 \text{ window})$ opening.

Fig. A-10 Finite element mesh for 0.1 aspect ratio wall with $3 \times 3 \text{ m}^2$ door plus $2(2 \times 1 \text{ m}^2)$ window) opening.

Fig. A-11 Horizontal stress distribution along the centerline,
Case 1: solid wall, linear temperature profile.

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