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Differential Movements and Stresses Arising in Masonry
Veneers of Highrise Structures.

University — Université

Carleton University, Ottawa

Degree for which thesis was presented — Grade pour lequel cette thèse fut présentée

Master of Engineering

Year this degree conferred — Année d'obtention de ce grade

1984

Name of Supervisor — Nom du directeur de thèse

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Differential Movements and Stresses
Arising in Masonry Veneers of Highrise Structures

by

© Gordon A. Fenton, B. Eng.

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
Carleton University
Ottawa, Ontario
April, 1984

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Abstract

In recent years a growing number of buildings have been making use of clay brick masonry as a veneer on the structure. Unfortunately, associated with this, the distress and failure of these veneers has become all too common lately. In the past, the design of the veneer system was based upon the assumption that the clay brick masonry itself had similar deformational properties as that of the supporting frame. An increasing body of experimental evidence, however, is showing that this is not so and that the different thermal, elastic, creep, and, in particular, moisture effects must be allowed for in the design.

The purpose of this study is twofold;

- 1) to derive from available experimental evidence a means to predict the differential movement (and resulting stresses) arising between a veneer and its supporting frame at any time during the life of a highrise structure, and
- 2) to evaluate the effect of various design parameters on the differential movements and/or stresses occurring in the exterior walls of loadbearing masonry and reinforced concrete structures.

Differential movements were calculated considering creep, shrinkage, thermal, and elastic strains as well as the freezing and moisture expansion strains peculiar to clay brick. To evaluate the effects of solar radiation on the temperatures developed in the brick veneer, an in-situ test was performed to measure surface and air cavity temperatures during the course of a year.

Two case studies were considered in this study. In the first, a loadbearing masonry structure, the location of the insulation was found to have a considerable effect on the annual cyclic differential movement between the veneer and the frame but a negligible effect on daily cyclic and long term differential movements. Fatigue life was found to be an important consideration in the design of the ties.

In the second case study, a reinforced concrete structure, the width of the expansion gap provided below each shelf angle was found to have the greatest effect on stresses developed in the veneer and stresses at lower levels were found to be largely independent of the stiffness of the overlying shelf angles. The location and thickness of the insulation was found to have a negligible effect on the maximum predicted stresses which were in all cases less than the axial compressive strength of the brick masonry. It is believed, however, that workmanship and stress concentration factors could still easily lead to cracking and failure of the veneer.

Acknowledgements

The author wishes to express his sincere thanks to;

Dr. G. T. Suter for his guidance and unending support under whose supervision this study was conducted.

The National Research Council of Canada for their generous financial support.

His wife, Terry, for her invaluable assistance during the final preparation of the thesis.

Professors W. Bowes and J. Goldak for their suggestions regarding heat flow.

Mr. H. Keller for his help, cooperation and practical suggestions.

Mr. A. J. Garwood, P. Eng., of Garwood-Leclair Inc., for his aid in the experimental phase.

Mastercraft Construction Ltd. for the use of their building in the experimental phase.

Dr. N. Holts for his help during the preparation of the thesis.

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Notation

A_{BL}	net cross-sectional area of the concrete block masonry within the tributary area of the supporting element
A_{CN}	net cross-sectional area of the column concrete
A_s	cross-sectional area of the structural steel column
A_{SBL}	cross-sectional area of the block masonry reinforcing steel within the tributary area of the supporting element
A_{SCN}	cross-sectional area of the column reinforcing steel
A_e	azimuth of the exterior wall normal measured clockwise from north
b	true thickness of the concrete column projecting beyond the inner wall face
b_{EFF}	effective thickness of the concrete column for heat flow calculations
C_l	sky clearness coefficient
CP	specific heat
d	true width of the concrete column measured parallel to the wall face
d_m	effective thickness of the concrete column for creep and shrinkage calculations
E_u	modulus of elasticity of the concrete block unit
E_C	modulus of elasticity of concrete
E_{grout}	modulus of elasticity of grout
E_{MBL}	modulus of elasticity of concrete block masonry
E_{MBR}	modulus of elasticity of clay brick masonry
E_{mort}	modulus of elasticity of mortar
E_{st}	modulus of elasticity of steel
E_{ugBL}	modulus of elasticity of ungrouted concrete block masonry
f	moisture correction factor for thermal conductivity

f'_u	compressive strength of the concrete block unit
f'_{br}	compressive strength of the clay brick unit
f'_c	compressive strength of concrete
f'_{cm}	mean compressive strength of concrete
f'_m	compressive strength of masonry
f'_{mort}	compressive strength of mortar
G_{sol}	normal solar radiant energy incident on a surface
GAP_{BL}	effective gap left between the upper concrete block masonry surface and the overlying slab
GAP_{BR}	effective gap left between the upper brick masonry surface and the overlying shelf angle
h_c	convective heat flow coefficient
h_r	radiative heat flow coefficient
k	coefficient of thermal conductivity
KSA	force required to displace the outer toe of the shelf angle one unit up or down relative to the supporting frame (shelf angle stiffness)
PC	building load acting on the supporting element(s)
$PVEN$	axial force transmitted to a brick panel from the panel above
R	ratio of modular to nominal vertical brick dimensions
R_M	ratio of modular to nominal vertical block dimensions
$R.H.\%$	relative humidity in percent
$STHT$	storey height
t	time or age
T	temperature
V	wind velocity
VSA	axial force transmitted to a brick panel from the upper shelf angle
$VYSA$	force required to produce a yield condition in the shelf angle when the outer toe is pushed up or down relative to the supporting frame

α	solar radiation absorptivity of a surface
α_M	coefficient of thermal expansion of concrete block masonry
α_{br}	coefficient of thermal expansion of clay brick masonry
α_{cc}	coefficient of thermal expansion of concrete
α_{st}	coefficient of thermal expansion of steel
β_a	first day plastic flow function for creep calculations
β_d	delayed elastic strain function for creep calculations
β_f	delayed plastic flow function for creep calculations
β_s	shrinkage function
ϵ	emissivity of a surface
ϵ_{bm}	brick unit moisture expansion strain
ϵ_{ca}	effective strain in the brick veneer due to corrosion expansion of the shelf angle
ϵ_{crBL}	creep strain in the concrete block masonry
ϵ_{crBR}	creep strain in the clay brick masonry
ϵ_{crCL}	creep strain in the column concrete
ϵ_{frBR}	freezing expansion strain in the clay brick masonry
ϵ_{moBR}	moisture expansion strain in the clay brick masonry
ϵ_{ms}	shrinkage strain in the brick veneer due solely to mortar shrinkage
ϵ_{S_1}	unadjusted ultimate concrete shrinkage strain
ϵ_{S_2}	effective thickness (d_m) correction factor for shrinkage in concrete
ϵ_{shBL}	shrinkage strain in concrete block masonry
ϵ_{shCL}	shrinkage strain in the column concrete
ϵ_{st}	thermal strain in steel
ϵ_{stBL}	thermal strain in the concrete masonry reinforcement

ϵ_{stCL}	thermal strain in the column reinforcement
ϵ_{thBL}	thermal strain in the concrete block masonry
ϵ_{thBR}	thermal strain in the clay brick masonry
ϵ_{thCL}	thermal strain in the column concrete
λ	ambient environment coefficient for the calculation of d_m
ρ	density
σ	stress
σ_{BL}	average axial stress acting on the concrete block masonry
σ_{BR}	average axial stress acting on the brick veneer
σ_{CONC}	average axial stress acting on the column concrete
σ_{SBL}	stress acting on the block masonry reinforcement
σ_{SCL}	stress acting on the column reinforcement
ϕ	specific creep function
ψ_d	delayed elastic strain coefficient for the calculation of creep
ψ_{f_1}	relative humidity flow coefficient for the calculation of creep
ψ_{f_2}	effective thickness flow coefficient for the calculation of creep

Chapter 1

Introduction

1.1 General

Throughout history, the appearance of a building has been of overriding importance in its design. At one time the design and construction of a building was carried out entirely by the architect, whose primary goal was to produce an aesthetically appealing structure. An economical strength design was of secondary importance since labour and materials were relatively inexpensive. Building shells were typically composed of a single material, for example, wood or stone block, which simultaneously served to support the structure, provide cover, and present a pleasing appearance.

As time went on, the need to economise in the use of both labour and materials became more important. Strength design became more sophisticated and efficient with the advent of, first steel, and then reinforced concrete structures, in which the amount of labour and material required to produce a building was decreased significantly. However, steel and concrete frames were usually considered unsightly and it was deemed desirable to cover them with an attractive veneer.

Now brick masonry is widely used as a veneer for hotels, schools, office, and residential buildings since it provides aesthetic appeal, durability, heat and sound insulation, as well as fire resistance. In loadbearing masonry structures the veneer is a non-loadbearing wall supported at the foundation and tied to the loadbearing masonry at regular intervals by flexible metal ties. In steel and concrete frame structures, on the other hand, the non-loadbearing veneer is generally supported at every level either by resting directly on the slab or on a steel shelf angle which in turn is fastened to the slab. Again, to provide lateral support, the veneer is tied at regular intervals to the backup wall or infill panel.

Unfortunately, designers in the past often overlooked the sometimes marked differences between the deformational properties of the veneer and its supporting frame. As a result, cracking and failure of the veneer has become all too common recently. At best these failures are unsightly and greatly diminish the building's serviceability and at worst they represent a severe hazard.

The original cost of a veneer is a significant portion of the total building cost and with the additional difficulty of refacing a building already in use, the cost of repairing a faulty veneer can be staggering.

In general, the problems associated with the veneer arise from differential movements between the veneer and the frame. The differential movement could result in over-stress or fatigue of the ties, particularly at the upper levels of a loadbearing masonry structure, or in a transfer of building loads to the veneer. In this report, a method of predicting the magnitude of the differential movements as well as the resulting stresses in the structural elements (veneer and frame) will be presented. This method will be applied to two case studies, one a loadbearing masonry structure, and the other a reinforced concrete structure, and some recommendations will be made.

1.2 Literature Review

1.2.1 Thermal Analysis

The general relationships governing steady-state heat flow through a wall section were derived from work by Hutcheon (52), who applied these relationships specifically to building skins. The form of these relationships is in agreement with those presented by Chapman (20), Jakob (53), and Eckert et al (31) for steady state heat flow through a multilayer wall. Although Hutcheon includes radiative heat flow coefficients in his analysis, no attempt is made to include the effect of solar radiation. This is considered justified because solar radiation is a transient effect which has been considered in this report using a transient heat flow analysis. However, if only a steady-state analysis were to be used, it would be best to include the effect of solar radiation when determining temperatures under extreme conditions. Grimm (39) presents a method to calculate the average temperature of a brick masonry wall subject to solar radiation. Grimm's analysis appears to be reasonable except that it can only be applied to a wall composed of a single layer of brick masonry. The air temperature just inside the inner surface of the masonry must be known and cannot be dependent on the brick temperature, as it would be if an inner layer of blockwork and/or insulation were present.

The theory describing transient heat flow is presented by several authors (20,31,53,79). Dusiinberre (29,30) proposes a simple, accurate numerical solution to the calculation of transient temperatures. The wall sandwich is broken into a series of finite elements, the future temperature of each (after a time interval Δt) being calculated on the basis of the present net heat flow into each. The solution is explicit, requiring only one equation to calculate this future temperature, but becomes unstable for time intervals which are too large. Dusiinberre suggests a criterion for the determination of the maximum time interval which will yield a convergent solution. As the time interval decreases below the maximum allowable value, the accuracy of the solution increases at the expense of a greater number of calculations. The only disadvantage to the proposed method is that the maximum time interval may be too small, requiring an inordinate number of calculations (for instance, if $\Delta t = 1$ sec, 86 400 calculations will be required for each element over a 24 hour period). However, this is not a serious limitation when the calculations are performed by computer.

An alternative numerical solution, which avoids the instability problem of the explicit solution, is described by both Chapman (20) and Eckert et al (31). It is an implicit formulation which calculates the future temperature of an element based on its present temperature and the future net heat flow into it. To proceed from one time step to the next, the future temperatures of all elements must be known simultaneously, and thus a set of simultaneous equations (one for each element) must be solved at each time interval.

The implicit method was rejected for the purposes of this report purely on the basis of its complexity. The explicit procedure suggested by Dusinberre has the potential of involving more calculations, but its simplicity outweighs this possible drawback with little or no sacrifice in accuracy.

Gughelmini (43) developed a mathematical model to calculate the amplitude and phase lag of a prescribed heat wave travelling through a wall section using a Fourier analysis. The wall sections considered consisted of 120 mm of brick veneer, an air cavity, 60 mm of mineral wool, and a final layer of 8 mm of asbestos-cement. In general the phase lag between exterior and interior air temperatures was found to be about 8 hours. The amplitude of the interior temperature varied from about 4% (for insulation outside the brick) to 22% (for insulation inside the brick) of the applied heat wave amplitude.

The solar radiation incident on a surface is discussed in detail by ASHRAE (5) including the relationships required to calculate its magnitude. Hooper et al (47) provide measured radiation values, taken at locations in Canada, that show reasonable agreement with those predicted by ASHRAE.

The thermal properties, conductivity, density, specific heat, emissivity, and absorptivity of common building materials were obtained from many different sources. The thermal conductivities of brick, for various densities and types, are given by BIA (12), Chapman (20), Fiorato et al (36), Lewis et al (67), Özisik (79), and Plummer (85). Although moisture content has a significant effect on the thermal conductivity of most building materials, none of these sources indicated what the moisture content of the brick was at the time of testing. Equations derived from Jakob's (53) observations, relating thermal conductivity with density and moisture content, show good agreement with tested solid brick thermal conductivity values, for moisture contents between 2 and 5 percent by volume. The relationships derived from Jakob are therefore recommended if actual thermal conductivity values are not known.

The specific heat of concrete brick was given to be $890 \text{ J/kg}^\circ\text{C}$ by Lewis et al (67), and fire clay brick was observed to have a specific heat between about 840 to $960 \text{ J/kg}^\circ\text{C}$ by Chapman (20) and Özisik (79). The emissivity and absorptivity values for brick masonry presented by ASHRAE (5) agree well with values presented by Grimm (39), Eckert et al (31), Chapman (20), Siegel et al (101), General Electric (37), Reagan (88), and Latta (61). Kasten (57) gives the absorptance of brick masonry to be 0.94 which is much higher than the maximum value of 0.85 given by ASHRAE. Since this so-called 'absorptance' value is also used in an equation which would normally use emittance, perhaps Kasten has simply confused absorptance for emittance. The emissivity of brick masonry

is commonly taken as 0.90 and the absorptivity ranges from 0.30 to 0.85 depending on the brick colour.

There is some question as to how representative the commonly accepted emissivity and absorptivity values are of 'in-situ' brick masonry. Chapman (20) gives the equilibrium temperature a body would reach in solar-irradiation environment to be;

$$T = \left[\left(\frac{\alpha}{\epsilon} \right) \left(\frac{I_D}{\sigma} \right) \right]^{0.25} \quad (^\circ K) \quad (1.1)$$

where,

α = solar radiation absorptivity of the body in question

ϵ = emissivity

I_D = solar radiation incident on the body (W/m^2)

σ = Stefan-Boltzmann constant

= $5.6696 \times 10^{-8} \quad W/m^2 \cdot ^\circ K^4$

For a south facing vertical surface in the Ottawa area, the incident solar radiation at noon on July 1 was found to be $501 W/m^2$. For an emissivity of 0.90 and an absorptivity of 0.70, Equation (1.1) predicts an equilibrium temperature of $15^\circ C$. This is much lower than the observed typical summer temperature values (see Section 2.2.1) of about $44^\circ C$. Bergquist (9) gives maximum brickwork temperatures of $46^\circ C$ to $48^\circ C$ for brown brick. Stephenson (103) expects wall surface temperatures to range between $60^\circ C$ and $90^\circ C$. Latta (62) predicts a maximum temperature rise of $23^\circ C$ above ambient or a brick temperature of at least $50^\circ C$.

Kirchhoff's law states that under equilibrium conditions the emissivity of a surface must be equal to its absorptivity. Under these conditions, Equation (1.1) yields an equilibrium brick temperature of $34^\circ C$. This value is still somewhat lower than expected and so it appears that the absorptivity of the brickwork under 'in-situ' conditions should actually be greater than its emissivity. Note that for an absorptivity of 0.85 and an emissivity of 0.70, Equation (1.1) predicts an equilibrium brick temperature of $49^\circ C$ which is more reasonable.

The thermal conductivity of concrete blockwork was derived primarily from work performed by NCMA (72). They did a thorough study which included the effects of block size and density, as well as the presence of insulation in the cores. Jakob's (53) observations on the effect of moisture content on the thermal conductivity of building materials has been applied to concrete blockwork in this report. Sablin (93) gives an estimate of the moisture content of blockwork as a function of the ambient relative humidity.

The specific heat of dry, normal weight concrete block is given by ASHRAE (5) to be $0.92 kJ/kg^\circ C$, and $0.88 kJ/kg^\circ C$ for light weight concrete block. ASHRAE also gives the emissivity of concrete blockwork as ranging from 0.85 to 0.95. The absorptivity of blockwork ranges from 0.55 to 0.61, according to Reagan (88).

Neville (78) presents values of thermal conductivity for normal and light weight concrete of various densities at two different moisture contents. These observations agree well with those by ASHRAE (5), Hutcheon (52), and General Electric (37). The variation of thermal conductivity with moisture content also shows good agreement with that predicted by Jakob (53).

The specific heat of oven dry concrete is estimated by ASHRAE (5) to be $0.92 \text{ kJ/kg}^\circ\text{C}$ and by General Electric (37) to be $0.85 \text{ kJ/kg}^\circ\text{C}$. Neville (78) expects the specific heat of concrete to lie between 0.84 and $1.17 \text{ kJ/kg}^\circ\text{C}$. The emissivity of concrete ranges from 0.85 to 0.95 and its absorptivity from 0.50 to 0.70 , according to ASHRAE (5).

The thermal properties of insulation were obtained from ASHRAE (5) and General Electric (37) for a variety of insulation types. The air cavity properties are presented in detail by Hutcheon (52) with the specific heat of air obtained from Chapman (20).

ASHRAE (5) provides a comprehensive method to calculate the amount of solar radiation energy incident on a surface, at any time, at any location on the earth. To simplify the procedure, only the Northern hemisphere is considered here. ASHRAE also provides reflectance coefficients for various foregrounds.

1.2.2 Prediction of Movement

Concrete

The creep properties of concrete have been investigated by a large number of researchers over the years. Few, however, have considered the effect of the age of the concrete at loading and the effect of incremental loading. Fintel and Khan (35) produced a set of design charts which can be used to predict the creep of plain concrete due to incremental loading. The effect of the reinforcement restraint is included in a correction factor.

The method proposed by Fintel and Khan is excellent if the designer wishes to calculate the creep in a column some time after construction is completed (when the only restraint is provided by the reinforcement). However, a significant portion of the column deformation often takes place during construction and it would be desirable to know the creep strain during this period. Fintel and Khan provide estimates of the specific creep (creep strain per unit stress) for a minimum load duration of 30 days. Thus, if the load increments are being applied at intervals less than 30 days and the magnitude of the creep strain is desired at the end of each interval, the proposed method becomes difficult to apply. The creep strain is required at the end of each interval to allow the effect of restraint by the brick veneer and/or infill blockwork as well as the reinforcement to be included in the analysis.

The method proposed by Swamy et al (108) to predict the creep of concrete is very similar to Fintel and Khan's proposal, and is perhaps somewhat superior as it includes the effect of relative

humidity on creep. However, only the ultimate creep strain can be determined, which is not suitable for this report.

Grimm (40) takes the effect of incremental loading into account, in his prediction of concrete creep, using a correction factor based on the time required to build each level and the number of levels above the level considered. Grimm also includes the effect of the gradual increase of concrete stiffness with time. The only restraint to creep considered is that offered by the reinforcement. Grimm's analysis can only be used to calculate the creep at times after completion of the structure (the number of levels above the level considered is used to determine the time of loading and must be fixed for each level).

The best method found to predict the creep of concrete at any time was developed by the Comité Européen du Béton (23). This method was recommended by Neville et al (77) and is based on the principle of superposition which can be stated as follows; "For average stress conditions, the strains due to creep are assumed to be a linear function of the stresses, and the creep strains due to part application of the stress, applied at different times, are considered additive" (24). The proposed method includes the effects of changing concrete stiffness, relative humidity, and several other parameters and allows the prediction of creep strain at any time due to any stress history. Neville (77) estimates that creep recovery is only about 2/3 that predicted by the principle of superposition. Peacock (81) indicates that the curing method can have a profound effect on creep. However, the extent of this effect was not quantified.

C.E.B. (23) also provides a method to predict the shrinkage strain in concrete at any time. The values of ultimate shrinkage strain obtained using C.E.B. procedures of around 450×10^{-6} mm/mm, tend to be somewhat lower than the ranges presented by Grimm (40), Neville (78), Hobbs (46), and the ACI Committee 209 (3) of around 550×10^{-6} to 1260×10^{-6} mm/mm. Baker (6) recommends a value for the ultimate shrinkage of concrete of 300×10^{-6} mm/mm, which is lower yet. The Building Research Establishment (17), Sahlin (93), and Winter (117) all present ranges in the expected ultimate shrinkage of concrete of around 200×10^{-6} mm/mm to 800×10^{-6} mm/mm which agrees well with the C.E.B. predictions.

Grimm (40) presents a relationship to predict the shrinkage of concrete at any time which involves several concrete property parameters that are not considered by C.E.B. (such as the slump, air content, cement content, aggregate ratios, etc. of the concrete considered). Although this relationship may be superior to the C.E.B. proposal, its complexity may be prohibitive to the average user.

The modulus of elasticity of concrete is given by the Canadian Standards Association Code A23.3-M77 (18) to be;

$$EC = 0.043w^{1.5}\sqrt{f'_c} \quad (MPa) \\ \approx 5000\sqrt{f'_c}$$

whereas C.E.B. (24) predicts the mean elastic modulus of normal weight concrete at 28 days to be;

$$EC = 11875\sqrt[3]{f'_c} \quad (MPa)$$

which tends to be 25 to 45 percent higher than the values predicted by the CSA for concrete strengths between 40 and 20 MPa. C.E.B. also provides the means to determine the variation of the elastic modulus with time, which differs from Grimm's (40) variation with time function by only about 4 percent for 1000 day old concrete

Emanuel and Hulsey (32) found that the coefficient of thermal expansion of concrete depends on the properties of its constituent materials as well as its age. A relationship is presented to predict the coefficient of thermal expansion of concrete which shows reasonably good agreement with observations by Grimm (40), Neville (78), and Berwanger et al (10). Grimm (40) provides a simplified relationship, depending on just the aggregate type, and aggregate to cement weight ratio, to predict the coefficient of thermal expansion of concrete

Clay Brick Masonry

One of the first thorough studies into the creep of clay brick masonry was performed by Poljakov (86) in 1962. A series of specimens was subjected to incremental loading. Poljakov found an almost linear relationship between the applied stress and creep strains for stresses not exceeding 60 percent of the failure stresses and developed a creep-time relationship which included the age at loading effect. The creep strains predicted by Poljakov's relationship tend to be considerably lower than those observed by Warren (115) for brickwork loaded at 28 days. This discrepancy may be due in part to different brick unit and mortar characteristics between the two countries, or to misunderstanding on the part of the author as to how the ultimate strength of brickwork is defined in the Russian system. Arutjunjan (4) applies Poljakov's relationships to some practical problems in the change of reinforcement stress due to creep of the brickwork.

Lenczner has been a major contributor to the study of brick masonry creep over the years (63,64,66). In their latest paper, Warren and Lenczner (115) develop a creep-time relationship based on Ross's hyperbolic equation,

$$\epsilon_c = \frac{t}{a + bt}$$

which shows good agreement with the creep strains observed in a series of test walls. However, the proposed method includes no correction factor for the age of the brick masonry at loading. In the calculation of the coefficients a and b , Warren and Lenczner differentiate between bricks laid wet or dry, to reflect the effect of suction on the mortar. The author believes that North American brick, laid wet or dry, most closely resembles the 'wet' British brick in its suction properties.

Grimm (40) includes the effect of age at loading, as well as the change in applied stress with time, in his proposed creep-time function. The rate of creep, for brick masonry loaded after 28 days, is found by Grimm to be substantially larger than that predicted by Poljakov (86) or by the C.E.B. (23) for concrete. Since BIA (13) reports that most of the creep takes place within the mortar joint, it would be expected that the rate of creep for loading at different times approach that predicted by the C.E.B. for concrete. In this respect Poljakov's relationship appears most reasonable. For loading at 28 days, Grimm's predicted creep strains show good agreement with

Lenczner's observations

The first reference to the moisture expansion of clay brick units was made in 1931 by Palmer (80) who carried out a comprehensive study on the volume changes in bricks and mortars due to variations in moisture contents. In 1958 Hosking and Hueber (50) developed a relationship between moisture expansion of clay brick masonry and time (which was later shown to be incorrect due to errors in measurement) and found that the rate of expansion increased with increasing ambient temperature. Further work by Hosking and Hueber (49) indicated that there was no practical relationship between the accelerated steam expansion test (used by McDowell and Birtwistle (69)) and expansion observed under natural exposure. However, McDowell and Birtwistle (69) found a good correlation between the accelerated test results and expansion observed under natural exposure. The amount of moisture expansion was found to vary with the logarithm of time which is in agreement with the findings of Grimm (40) and Smith (102). The effect of ambient temperature was discussed by Beard et al (8) and appears to be in agreement with the findings of Hosking and Hueber (50). Jessop et al (56) discuss the effects of firing temperature on the magnitude of moisture expansion. A considerable difference between rates of moisture expansion was observed by Ritchie (90) for varying ambient relative humidities. Schellbach (95) indicates that moisture expansion is not isotropic due to alignment of the clay particles upon forming, expansion being greatest in the direction of extrusion.

Grimm's (40) relationship between moisture expansion and time shows relatively poor agreement with Ritchie's (90) observations. However, if the base 10 logarithm is used to relate expansion with time, instead of the natural logarithm, a much closer agreement is obtained. To estimate the moisture expansion of a brick masonry wall, Grimm also includes the effects of mortar shrinkage. Pieper and Hage (82) observed negligible moisture expansion of clay brick masonry in two test walls, which they concluded was due to shrinkage in the mortar joints. Wyatt (118) estimates the amount of reversible moisture expansion of clay brick masonry, due to moisture variations encountered on site, to be about 0.01 percent. This factor is difficult to predict, though, because of the uncertainty as to how moisture content varies with ambient conditions.

Values of the coefficients of thermal expansion of brick units and brick masonry were obtained from many sources (2,6,9,13,17,68,70,75,80,85,91,96,99,106). Grimm (40) presents a relationship between the net coefficient of thermal expansion of brick masonry and the individual coefficients of the brick unit and mortar.

Davison (28) performed an exhaustive study on the magnitude of the strains experienced by clay brick upon freezing. The bricks were saturated prior to freezing, though, so there is some question as to how representative his results are to freezing strains experienced in a normal exposure condition.

To determine the elastic modulus of clay brick masonry, Powell and Hodgkinson (87) studied the stress-strain curves derived from axial compressive tests on several brick masonry pillars. The calculated tangent and secant moduli differed widely and no substantial conclusion was offered. Lenczner (84) found that the elastic properties of brick masonry were best predicted from the

compressive strength of the brick masonry, which is in agreement with work by Eskenazi (33). Eskenazi's predicted elastic moduli shows good agreement with those predicted by Lenczner, and, although Lenczner observed that the geometry of the member has no consistent effect on the elastic modulus, Eskenazi goes on to present an alternative formulation which takes into account the wall thickness

Jessop et al (55) and Shrive et al (100) describe a method of determining the net elastic modulus of brick masonry from the elastic properties of the individual components - mortar and brick. However, the method appears overly complicated without a significant increase in accuracy. Sceviour (94) estimates, from Lenczner's work, the elastic modulus of mortar to be,

$$E_{mort} = \frac{5000 \sqrt{f'_{mort}}}{4 \frac{l}{c} + 1}$$

where $\frac{l}{c}$ is the lime to cement ratio

Grimm (40) predicts the elastic modulus of brick masonry, based on elastic moduli test results, as a function of time. An alternative formulation is also described for use if the mortar and brick material properties are known. Both relationships appear to be superior to all others considered.

Concrete Blockwork

Lenczner (65) studied creep strains in low-strength concrete blockwork piers and found the ratio of maximum load strain to instantaneous strain to vary between 4.3 and 7.6. The creep in the mortar joints was found to exceed that in the block unit by a factor of 4 to 5. Tests carried out by NCMA (76) on high strength block show the ratio of maximum load strain to instantaneous strain to vary between 1.5 and 2.5. Since most concrete block used in North America has much higher strength than that used by Lenczner, his observations have questionable value in North America.

Tatsa et al (111) found the specific creep (creep per unit stress) in hollow concrete block to vary between 7.7×10^{-6} to 9.3×10^{-6} mm/mm/MPa at an applied load of 45 percent of the ultimate strength. These values are far lower than the specific creep obtained from NCMA (76) observations of about 4.8×10^{-5} mm/mm/MPa for concrete blockwork. Buchanan (16) observed specific creeps in concrete blockwork between 8.8×10^{-5} to 9.1×10^{-5} for ungrouted masonry and between 9.5×10^{-5} to 12.3×10^{-5} mm/mm/MPa for grouted masonry.

Lenczner (65) develops a relationship to describe the creep of concrete blockwork as a function of time, however it shows poor agreement with the NCMA observations and cannot be applied to an incremental loading situation.

The causes of concrete blockwork shrinkage are discussed in some detail by Jessop (58) and Sturgeon (106). NCMA (73) gives values of shrinkage of concrete block units for various aggregate types ranging from 0.013% to 0.090% with an average value of 0.038%. Sahlin (93) presents a range in shrinkage values from 0.04% to 0.08% and indicates how shrinkage varies with ambient relative humidity. Observations made by Copeland (25) on moisture shrinkage are in good agreement with

those by NCMA (73) BIA (13) estimated that 25 to 30 percent more shrinkage would occur in a wall than in the block units, whereas Hedstrom (45) observed somewhat less shrinkage in the walls than in the block units (which he attributed to restraint by the mortar).

Allan (1), Bryson and Watstein (15), and Valore and Kuenning (114) all agree that structural shrinkage is reduced with increased storage period. HHFA (51) found that rewetting the block resulted in an expansion of roughly 17 to 33 percent of the shrinkage which had already occurred.

Values of the coefficient of thermal expansion of concrete block were obtained from Sturgeon (106), Jessop (56), BRE (17), and BIA (13) and range from $3.8 \times 10^{-6}/^{\circ}\text{C}$ to $12 \times 10^{-6}/^{\circ}\text{C}$ with an average value of about $9.0 \times 10^{-6}/^{\circ}\text{C}$.

Work by Sturgeon (105) indicates that the elastic modulus of grout in contact with concrete block averages only about 56 percent of the value determined from a standard moist cured cylinder test. Both Sturgeon (105) and Eskenazi (33) have developed relationships between the elastic modulus of ungrouted concrete blockwork and masonry strength, based on tests performed in Canada. Sahlin (93) suggests an alternative method of determining the elastic modulus of ungrouted blockwork based on the individual elastic moduli of the block and mortar. The mechanical properties of mortar have been studied by the Colorado Masonry Institute (21,22).

Based on his own findings, Sturgeon (105) also proposes a method to find the elastic modulus of partially or fully grouted concrete blockwork. Baynit (7) looks at the effect of grout on the compressive strength of concrete blockwork. The CSA Code S304 (19) gives the elastic modulus of concrete blockwork to be equal to $1000f'_m$ based on a characteristic compressive strength. Fattal and Cattaneo (34) found the elastic modulus of concrete blockwork to vary between $675f'_m$ and $1200f'_m$ with an average of $850f'_m$ based on a mean compressive strength. Some test data, indicating the variation of elastic modulus with time, is presented by NCMA (76).

Ties and Shelf Angles

Yura (119) develops a method in which the shear stiffness and yield strength of the shelf angle is determined as a function of its geometry and the connection (between the frame and shelf angle) strength. However, the proposed method is quite complex and was used in this report only to obtain typical stiffness and yield strength values. Yura includes in his paper a simple example of the compressive forces arising in a masonry veneer (supported on shelf angles) due to differential movements. In his example, the frame was assumed to be infinitely stiff in the vertical direction. Since the frame in general actually has a stiffness of the same order as that of the shelf angle (within the tributary area), Yura is considerably overestimating the compressive forces arising in the veneer.

The effect of shelf angle corrosion on the vertical movement of the supported veneer is discussed briefly by Grimm (40) who also presents a relationship between the growth of the corrosion products and time. Stetina (104) estimates the average rate of steel surface corrosion as 0.01 mm per year for a relatively dry surface to 0.13 mm per year when the surface is exposed to generous amounts

of water and oxygen. The effects of protective paints or an acidic environment must be considered by the designer on an individual basis.

There are numerous types of ties used in construction to secure the masonry veneer to its backup. In general these ties can be considered to have negligible shear stiffness. All too often, though, the cavity behind the veneer is practically filled with mortar droppings which can provide significant restraint to differential movement. Unfortunately no research has been done at this time on this particular topic.

Bergquist (9) takes a close look at the effect of differential movements on the fatigue life of a tie. The expected life span of a tie can be determined easily from Bergquist's work if the daily thermal movement between the veneer and the backup is known.

Although corrosion is a potential source of tie failure in North America, it is not considered in this report.

1.2.3 Full Scale Experimental Evidence

Temperature Studies on Building Skins

Two parties, Gujral et al (44) and Kusuda et al (58), performed transient thermal tests on a massive, one room, externally insulated masonry building. In both cases the primary goal was to determine the required cooling loads and so no element temperature data was given. Bergquist (9) observed the temperature variation of several brick masonry panels in Sweden and gives maximum wall temperatures (at a point half way between wall surfaces) of 37°C to 48°C depending on the wall colour. Dexter et al (27) measured the surface temperature of 12 walls (of different mass, colour, orientation, insulation, and climatic conditions) of 10 houses in New Mexico on a half hourly basis. The surface temperature data is given over a six day period for a south facing, dark coloured, light weight concrete masonry wall. The temperature varies sinusoidally with a maximum surface temperature of 31°C and a maximum amplitude of 16°C . This temperature data does not appear to be extreme and in fact was only used by Dexter et al to verify a mathematical model they had developed.

Building Movements

Russell and Corley (92) instrumented a 262 m high concrete structure (Water Tower Place) to measure the time dependent deformations in columns and walls at several different levels. Two years after completion of the structure, maximum column strains were measured to be about 1.4 mm/m . Russell et al then attempted to predict the time dependent column strains taking into account loading history, column size, reinforcement, and concrete properties and reasonably good agreement was obtained with the observed values.

Swamy and Potter (110) investigated the long-term movements in an eight storey reinforced concrete structure. The strains in several columns were measured both during construction and after completion of the structure. Almost one year after completion, the total strain in an interior column at the second storey averaged 0.59 mm/m and at the eighth storey 0.24 mm/m .

Swamy and Arumugasaamy (109) report on the long-term movements observed in two reinforced concrete structures, one of which was discussed in greater detail by Swamy and Potter (110). The other was a six storey structure in which several columns in the first storey and basement were instrumented. After $7\frac{1}{2}$ years, the observed total strain (excluding thermal strains) in the basement columns averaged about 0.45 mm/m and at the first storey about 0.60 mm/m .

1.2.4 Veneer Distress

The common modes of failure of brick masonry veneer and many of their causes are illuminated by Plewes (83,84) and Suter (107). Due to construction difficulties, the mason often places the masonry ties too far away from the shelf angles, leaving the veneer unsupported over heights as much as 1.5 m (except for some frictional restraint at the shelf angle). With the addition of some axial force, due to differential movement and inadequate expansion joints, this will often lead to local buckling of the veneer at loads much less than that predicted by the compressive strength of the brick masonry (see Figure 1.1).

Spalling of the brick veneer may arise from several sources. The gap between the outer surface of the veneer and the toe of the shelf angle, as shown in Figure 1.2, is usually filled with mortar. This will result in a concentration of the stresses transmitted through the veneer from above. The stress at the toe of the shelf angle can be many times the average axial stress in the veneer and can easily lead to crushing of the mortar and/or spalling of the brick face.

The shelf angle itself, upon deforming, can contribute to spalling and buckling failures. When differential movements take place the shelf angle may be forced to rotate and deform, transferring load directly to the edge of the masonry, as shown in Figure 1.3. This may lead to cracking and spalling of the brick along the loaded edge and also to buckling of the veneer due to the large eccentricity of the applied force.

Suter (107) predicted that the mode of failure of the stressed veneer is largely dependent on the amount that the veneer overhangs the toe of the shelf angle (see Figure 1.4). Seddeik (98) considers this effect and concludes that although in most cases the average axial stress in the veneer does not exceed f'_m , the practice of mortaring in the joint at the toe of the shelf angle or providing an excessive overhang will often lead to distress or failure of the veneer.

Glanville (38) looks at the structural implications of thermal effects in loadbearing masonry structures. The loading and unloading of an interior wall due to the thermal movements of the exterior wall is predicted and some of the results indicate extensive horizontal cracking of both the exterior and interior walls, particularly at the upper levels.

Figure 1.1 Typical cavity wall and veneer construction showing one manner of buckling (from Plewes, 83).

Figure 1.2 Bed joint in the veneer at the shelf angle (from Plewes, 83).

Figure 1.3 Rotation of the shelf angle due to relative movement of the frame (from Plewes, 83).

a. Narrow Joint

b. Normal Joint

c. Wide Joint

Figure 1.4 Veneer failure conditions (from Suter, 197)

1.3 Scope and Objectives

To establish the stresses and potential problems arising from differential movement between a frame and its veneer, it is necessary to first determine the magnitude of these movements. This involves estimating the total vertical deformations occurring in the concrete, steel, blockwork, and brick veneer, as a function of time, using suitable mathematical formulae or relationships, and comparing them. Once the differential movements are known, the restraining forces between the individual elements can be calculated, allowing the total axial stress acting on each element to be found. In this report, all forces are assumed to act axially with no eccentricity.

The resulting analysis will be applied to two case studies (one a loadbearing masonry structure and the other a reinforced concrete structure) to evaluate the effect that various building parameters have on differential movement and element stresses. In particular, for loadbearing masonry structures, the objectives of this report are to;

- 1) estimate the effect of solar radiation on the temperatures experienced by the brick veneer,
- 2) evaluate the effect of insulation position and efficiency on the differential movement occurring between the brick veneer and the frame,
- 3) comment on the probability and causes of failure or distress arising in the brick veneer.

In reinforced concrete structures, in which the brick veneer is supported at regular intervals by steel shelf angles, the objectives of this report are to;

- 1) evaluate the effect of the assumed ultimate moisture movement parameters (ie., concrete shrinkage and clay brick moisture and freezing expansion) and ambient thermal conditions on the axial stress developed in the brick veneer,
- 2) evaluate the effect of time (or building age) on the axial stress developed in the brick veneer,
- 3) evaluate the effect of the thickness of the expansion gap, provided under each shelf angle, on the axial stress developed in the brick veneer,
- 4) evaluate the effect of the assumed stiffness and yield point of the shelf angle on the axial stress developed in the brick veneer,
- 5) evaluate the effect of insulation position and efficiency on the axial stress developed in the brick veneer,
- 6) comment on the probability and possible causes of failure or distress arising in the brick veneer.

Chapter 2

Thermal Analysis

2.1 Introduction

The temperatures of the various elements in a wall section, under any particular conditions, are of interest to the designer for at least two reasons,

- 1) the resulting thermal strains in the wall elements may lead to stress or fatigue problems if the differential movements are not considered in the design, and
- 2) to determine the thermal efficiency of the wall section

Although the latter has been receiving more attention recently due to rising energy costs, this study deals strictly with the former. The thermal strain found in a structural material is often a major source of movement and can be larger than that due to creep, moisture, or elastic strain, particularly if the element is subject to large fluctuations in temperature. Because of this potentially significant contribution to structural movement, it is important that the thermal strain in each element be predicted reasonably accurately.

Thermal stresses which may occur in building elements are generally a result of differential rather than absolute movements, thus it is of primary interest to find the maximum temperature differences between elements. To clarify this concept, consider a simple example of an exterior wall consisting of an outer wythe of brick masonry and an inner wythe of blockwork, as shown in Figure 2.1.

During the course of a day, the ambient temperature can be assumed to vary sinusoidally. As the ambient temperature rises, heat begins to flow into the brick. Depending on the thermal mass of the brick, this heat wave will take some time to actually travel to the inner surface of the brick masonry. Thus the maximum temperature of the brick masonry will occur some time after the maximum brick surface temperature is achieved.

The heat wave continues to travel through the wall section at a speed dependent on the resistance and thermal mass of the material through which it passes. Because of thermal resistance, the amplitude of the heat wave will also decrease as it moves through the material. The temperature variation of the environment, brick masonry, and blockwork with time might typically appear as shown in Figure 2.2. The maximum temperature differentials (Δ_{max}) do not necessarily coincide with the maximum and minimum wall element temperatures.

Figure 2.1 Simple exterior wall section.

Figure 2.2 Variation of temperature with time of day (idealized).

To arrive at a reasonable estimate of the differential temperatures arising in any given wall section under known environmental conditions, there are three possible approaches,

- 1) to make a prediction on the basis of experimental evidence,
- 2) to use a steady state heat flow analysis, or
- 3) to use a transient heat flow analysis

Although experimental evidence exists (9,11,27,44,58,61,62,88,103) on the temperature variations observed in wall elements, it is not completely general and cannot be applied equally well to all situations

The steady state heat flow analysis is a good approach only if the environmental conditions do not change markedly over a period of several hours. However if these conditions do change rapidly (as in a hot, sunny summer day), the optimum solution will involve a transient heat flow analysis. Furthermore, if the transient heat flow model is calibrated against experimental evidence, it can be applied with confidence to almost all situations. The transient thermal analysis developed for this report is discussed in greater detail in Section 2.3

Besides the extremely hot days which require this complex transient analysis, there are other 'thermal' situations which are important to the calculation of instantaneous and long term movements. One of these is the minimum mid-winter temperature, which, depending on geometry and the position of the insulation, could lead to severe stresses. Since the coldest temperatures generally occur at night, this situation can be considered to be steady state, greatly simplifying the analysis

If stresses in the wall materials arise from the previously discussed thermal conditions, then they will in turn contribute to creep in the material. The total creep can be considered proportional to the average stress in the wall element and daily fluctuations can be ignored. Thus if the average daily or monthly temperatures lead to stresses, they must be included in the model. These thermal conditions can also be considered to be steady state.

In summary, there are three thermal conditions which are of interest in this report,

- 1) the maximum summer temperatures, which will be determined using a transient one-dimensional heat flow analysis,
- 2) the average temperatures at a particular time, which will be used to determine creep stresses and will be found using a steady state one-dimensional analysis, and
- 3) the minimum winter temperatures which, again, will be found using a steady state one-dimensional analysis.

2.2 Steady State Thermal Analysis

The steady state thermal analysis described in this section will be used to predict wall element temperatures due to average ambient temperatures as well as the minimum winter temperature. To simplify the mathematics involved, the steady state heat flow through a wall section will be assumed to be one-dimensional. This assumption creates two constraints;

- 1) The effects of thermal bridging must be taken into account by calculating an 'effective' conductivity of the layer in question. This means that the bridges and the adjoining material will be predicted to have the same temperature.
- 2) Since the heat flow lines will be assumed parallel, the temperature of a 'cooling fin' (for example, a column or return wall projecting into the interior) cannot be determined directly

A plane wall generally has parallel heat flow lines, but the actual heat flow through a wall with a column projecting into the interior will typically appear as shown in Figure 2.3

Figure 2.3 Heat flow and temperature distribution occurring in a wall with a projecting column.

If the wall section shown in Figure 2.3 was modeled as is in a one-dimensional steady state heat flow analysis, the temperature distribution would appear as shown in Figure 2.4

Figure 2.4 Temperature distribution in the projecting column predicted by a one-dimensional steady state heat flow analysis.

The one-dimensional analysis predicts four temperature drops occurring within the projecting fin, as compared to the real situation (Figure 2.3) in which only slightly more than one temperature drop occurs within the fin. This discrepancy can be solved by calculating an effective projecting thickness, b_{EFF} , such that the average temperature of the fin, as determined by the one-dimensional analysis, is equal to that in the real case. This calculation is discussed in detail in Appendix B

The basis of the steady state, one-dimensional heat flow analysis can be summed up in the following relationship;

$$Q = \left(\frac{A}{R} \right) \Delta T \quad (2.1)$$

where,

Q = rate of heat flow through the element (Watts)

A = area of the element transverse to the flow (m^2)

R = thermal resistance of the element ($m^2 \cdot ^\circ C/W$)

ΔT = temperature drop across the element ($^\circ C$)

If q is defined as the rate of heat flow through a unit area of the element (W/m^2), then;

$$q = \frac{\Delta T}{R} \quad (2.2)$$

which can be expanded to,

$$q = \frac{T_{ext} - T_{int}}{R_T} \quad (2.3)$$

where,

T_{ext} = exterior ambient temperature ($^{\circ}C$)

T_{int} = interior air temperature ($^{\circ}C$)

R_T = total wall thermal resistance including interior and exterior air film resistances ($m^2 \cdot ^{\circ}C/W$)

Figure 2.5 Example wall section.

If a wall section is considered as shown in Figure 2.5, then the temperatures can be determined as follows;

A) First the thermal resistance of each element must be determined;

1) Exterior Air Film (from Hutcheon, 52)

$$R_1 = \frac{1}{hc + hr} \quad (2.4)$$

where,

$$\begin{aligned} hc &= \text{convective heat flow coefficient (W/m}^2 \cdot ^{\circ}C) \\ &= 5.62 + 3.91V \quad \text{for wind velocity } V < 5 \text{ m/sec} \\ &= 7.34(V)^{0.8} \quad \text{for } 5 \leq V \leq 30 \text{ m/sec} \end{aligned}$$

$$\begin{aligned}
 hr &= \text{radiative heat flow coefficient (W/m}^2 \cdot ^\circ\text{C)} \\
 &= \epsilon_{br} \sigma (T_1'^2 + T_{ext}'^2) (T_1' + T_{ext}') \quad (2.5)
 \end{aligned}$$

T_1' = exterior surface temperature in $^\circ\text{K}$

T_{ext}' = ambient air temperature in $^\circ\text{K}$

V = effective wind velocity on the surface in question in m/sec

ϵ_{br} = emissivity of the exterior surface

σ = Stefan-Boltzmann constant

$$= 5.6696 \times 10^{-8} \text{ W/m}^2 \cdot ^\circ\text{K}^4$$

Note that the radiative heat flow coefficient hr is dependent on temperature. Since this, as well as several other of the element resistances, is dependent on temperature, not only must the initial temperature be estimated but the solution should be iterated to close in on the true values of resistance and, thus, heat flow.

2) Brick Veneer

$$R_2 = \frac{l_2}{k_{br}} \quad (2.6)$$

where,

l_2 = thickness of the veneer (m)

k_{br} = coefficient of thermal conductivity of brick ($\text{W/m}^\circ\text{C}$)

3) Air Cavity

$$R_3 = \frac{1}{hc + hr}$$

where,

hc = convective heat flow coefficient ($\text{W/m}^2 \cdot ^\circ\text{C}$)

hr = radiative heat flow coefficient ($\text{W/m}^2 \cdot ^\circ\text{C}$)

(see Appendix A)

4) Insulation, Blockwork, and Finish

$$R_{4,5,6} = \frac{l_{4,5,6}}{k_{4,5,6}}$$

where,

l_i = thickness of the element in question (m)

k = coefficient of thermal conductivity ($\text{W/m}^\circ\text{C}$)

5) Interior Air Film (from Hutcheon, 52)

$$R_7 = \frac{1}{hc + hr}$$

where,

$$hc = 1.87 (T_0 - T_{int})^{0.32} (H)^{-0.05} \quad (W/m^2 \cdot ^\circ C)$$

$$hr = \epsilon_f \sigma (T_0'^2 + T_{int}'^2)(T_0' + T_{int}') \quad (W/m^2 \cdot ^\circ K)$$

H = height of wall in m

T_0, T_{int} = surface and interior temperature in $^\circ C$

T_0', T_{int}' = surface and interior temperature in $^\circ K$

B) The total wall resistance can now be determined to be;

$$R_T = R_1 + R_2 + R_3 + \dots + R_7$$

C) Calculate the total heat flow through a unit area using Equation (2.3);

$$q = \frac{T_{ext} - T_{int}}{R_T}$$

D) Since the heat flow must be the same at any point through the wall sandwich, the individual temperatures can be found as follows;

$$T_1 = T_{ext} - q \cdot R_1 \quad (^\circ C) \quad (2.8)$$

similarly,

$$T_i = T_{i-1} - q \cdot R_i \quad (^\circ C) \quad (2.9)$$

See Appendix A for typical material thermal properties.

2.3 Transient Thermal Analysis

Transient heat flow is of interest to the designer because the maximum temperature differential between supporting elements may not occur at the same time as the peak temperature in either element. Furthermore, the analysis will consider the effect of changes in external ambient temperature and solar radiation energy on the temperatures of all layers in an exterior wall.

To simplify the model, the heat flow will be assumed to be one-dimensional, with the same resulting constraints discussed in Section 2.2. The wall section can now be represented as a bar element composed of the appropriate materials, as shown in Figure 2.6,

Figure 2.6 Bar element model of the wall section.

Figure 2.7 Transient heat flow finite difference model.

If the bar element depicted in Figure 2.6 is divided into a series of nodal elements (as shown in Figure 2.7), then the unsteady state nodal equation for, say, node #2 will be,

$$\left(\frac{k_{12}}{\Delta x_1} + \frac{k_{23}}{\Delta x_2} \right) (T_{1,j} - T_{2,j}) + \left(\frac{k_{23}}{\Delta x_2} + \frac{k_{34}}{\Delta x_3} \right) (T_{3,j} - T_{2,j}) = \left(\frac{M_2 \cdot CP_2}{\Delta t} \right) (T_{2,j+1} - T_{2,j}) \quad (2.10)$$

where,

- k_{lm} = coeff of thermal conductivity between nodes l and m ($W/m^\circ C$)
- M_i = mass of element i (kg)
- CP_i = specific heat of element i ($J/kg^\circ C$)
- Δt = time lapse between t_j and t_{j+1} (sec)
- Δx_i = thickness of element i (m)
- $T_{i,j}$ = temperature of element i during time interval j ($^\circ K$)

Note that the prime notation on the temperature in $^\circ K$ has been dropped since all the temperatures in this section are measured in the Kelvin scale

Making the further simplification that within any particular material Δx is constant, and expressing M_i as a function of the density ρ in kg/m^3 , Equation (2.10) can be reduced to,

$$\left(\frac{k_{12}}{\Delta x} \right) (T_{1,j} - T_{2,j}) + \left(\frac{k_{23}}{\Delta x} \right) (T_{3,j} - T_{2,j}) = \left(\frac{\rho_2 \cdot \Delta x \cdot CP_2}{\Delta t} \right) (T_{2,j+1} - T_{2,j}) \quad (2.11)$$

from which the future temperature at node #2 can be solved as follows,

$$T_{2,j+1} = \left(\frac{\Delta t}{\rho_2 \cdot \Delta x^2 \cdot CP_2} \right) \left[k_{12} T_{1,j} + k_{23} T_{3,j} + T_{2,j} \left(\frac{\rho_2 \cdot \Delta x^2 \cdot CP_2}{\Delta t} - k_{12} - k_{23} \right) \right] \quad (2.12)$$

To avoid divergence or instability in the iterative solution, Dusenberre (29) suggests that negative coefficients in front of the temperature terms be avoided. Since, by definition, the coefficients k_{12} and k_{23} are always positive, it is only the coefficient of $T_{2,j}$ which must be limited, such that,

$$\frac{\rho_2 \cdot \Delta x^2 \cdot CP_2}{\Delta t} \geq k_{12} + k_{23} \quad (2.13)$$

which in turn implies that the smaller the element, the more frequently a time step must be taken (ie Δt must also decrease)

Applying the finite difference concept directly to the bar element shown in Figure 2.6 will yield a finite difference model similar to that seen in Figure 2.8

Figure 2.8 Finite difference model of wall section

Including convective and solar thermal loading, the unsteady state nodal equations for node #1 can be written as follows,

$$\text{HEAT IN} = \epsilon_{br} \sigma T_{ext,j}^4 + \alpha_{br} \cdot G_{sol,j} + h_{c_{ext,j}} T_{ext,j} + \left(\frac{k_{12,j}}{\Delta x_{br}} \right) T_{2,j} \quad (2.14)$$

$$\text{HEAT OUT} = T_{1,j} \left[\frac{k_{12,j}}{\Delta x_{br}} + h_{c_{ext,j}} + \epsilon_{br} \sigma \cdot T_{1,j}^3 \right] \quad (2.15)$$

$$\text{HEAT STORED} = \rho_{br} C P_{br} \left(\frac{1}{\Delta t} \right) \left(\frac{\Delta x_{br}}{2} \right) (T_{1,j+1} - T_{1,j}) \quad (2.16)$$

where,

ϵ_{br} = total hemispherical emittance of brick

$T_{ext,j}$ = exterior ambient temperature at the time t_j ($^{\circ}K$)

α_{br} = fraction of solar radiation absorbed by the brick

$G_{sol,j}$ = normal solar radiant energy striking the surface at the time t_j
(W/m^2)

$h_{c_{ext,j}}$ = external convective heat transfer coefficient at the time t_j ($W/m^2 \cdot ^{\circ}K$)

Combining Equations (2.14), (2.15), and (2.16) as follows;

$$(\text{HEAT IN}) - (\text{HEAT OUT}) = (\text{HEAT STORED}) \quad (2.17)$$

allows the future temperature of node #1 to be solved;

$$T_{1,j+1} = C_1 \left[\epsilon_{br} \sigma T_{ext,j}^4 + \alpha_{br} \cdot G_{sol,j} + h_{c_{ext,j}} T_{ext,j} + \left(\frac{k_{12,j}}{\Delta x_{br}} \right) T_{2,j} + T_{1,j} \left(\frac{1}{C_1} - \frac{k_{12,j}}{\Delta x_{br}} - h_{c_{ext,j}} - \epsilon_{br} \sigma \cdot T_{1,j}^3 \right) \right] \quad (2.18)$$

where $C_1 = \frac{2 \Delta t}{\rho_{br} \cdot CP_{br} \cdot \Delta x_{br}}$ (2.19)

For stability, it is necessary that;

$$\Delta t \leq \frac{\rho_{br} \cdot CP_{br} \cdot \Delta x_{br}}{2 \left(\frac{k_{1,2,j}}{\Delta x_{br}} + hc_{ext,j} + \epsilon_{br} \cdot \sigma \cdot T_{1,j}^3 \right)} \quad \text{seconds} \quad (2.20)$$

For nodes 2,5,7,9, and 11 (ie. interior nodes), the following relationships apply;

$$\text{HEAT IN} = \left(\frac{k_{lm,j}}{\Delta x_m} \right) \cdot T_{l,j} + \left(\frac{k_{mn,j}}{\Delta x_m} \right) \cdot T_{n,j} \quad (2.21)$$

$$\text{HEAT OUT} = T_{m,j} \cdot \left[\frac{k_{lm,j}}{\Delta x_m} + \frac{k_{mn,j}}{\Delta x_m} \right] \quad (2.22)$$

$$\text{HEAT STORED} = \rho_m \cdot CP_m \cdot \Delta x_m \cdot \left(\frac{1}{\Delta t} \right) (T_{m,j+1} - T_{m,j}) \quad (2.23)$$

The subscript l refers to the preceding node, m to the node under consideration, and n to the following node. For example, $k_{lm,j}$ is the thermal conductivity between nodes l and m during the time interval j .

Using Equation (2.17), the future temperatures of each interior node can be found as follows;

$$T_{m,j+1} = C_2 \cdot \left[\left(\frac{k_{lm,j}}{\Delta x_m} \right) \cdot T_{l,j} + \left(\frac{k_{mn,j}}{\Delta x_m} \right) \cdot T_{n,j} + T_{m,j} \cdot \left(\frac{1}{C_2} - \frac{k_{lm,j}}{\Delta x_m} - \frac{k_{mn,j}}{\Delta x_m} \right) \right] \quad (2.24)$$

where $C_2 = \frac{\Delta t}{\rho_m \cdot CP_m \cdot \Delta x_m}$ (2.25)

For stability, it is necessary that;

$$\Delta t \leq \frac{\rho_m \cdot CP_m \cdot \Delta x_m^2}{k_{lm,j} + k_{mn,j}} \quad \text{seconds} \quad (2.26)$$

At the boundary of the air cavity (ie. nodes 3 and 4), the following relationships apply;

$$\text{HEAT IN} = \left(\frac{k_{lm,j}}{\Delta x_m} \right) \cdot T_{l,j} + hc_{mn,j} \cdot T_{n,j} + \epsilon_n \cdot \sigma \cdot T_{n,j}^4 \quad (2.27)$$

$$\text{HEAT OUT} = T_{m,j} \cdot \left[\frac{k_{lm,j}}{\Delta x_m} + hc_{mn,j} + \epsilon_m \cdot \sigma \cdot T_{m,j}^3 \right] \quad (2.28)$$

$$\text{HEAT STORED} = \rho_m \cdot CP_m \cdot \left(\frac{\Delta x_m}{2} \right) \left(\frac{1}{\Delta t} \right) (T_{m,j+1} - T_{m,j}) \quad (2.29)$$

giving,

$$T_{m,j+1} = C_1 \left[\epsilon_m \cdot \sigma \cdot T_{n,j}^4 + h c_{mn,j} \cdot T_{n,j} + \left(\frac{k_{lm,j}}{\Delta x_m} \right) \cdot T_{l,j} + T_{m,j} \cdot \left(\frac{1}{C_1} - \frac{k_{lm,j}}{\Delta x_m} - h c_{mn,j} - \epsilon_m \cdot \sigma \cdot T_{m,j}^3 \right) \right] \quad (2.30)$$

$$C_1 = \frac{2 \cdot \Delta t}{\rho_m \cdot CP_m \cdot \Delta x_m} \quad (2.31)$$

$$\Delta t \leq \frac{\rho_m \cdot CP_m \cdot \Delta x_m}{2 \left(\frac{k_{lm,j}}{\Delta x_m} + h c_{mn,j} + \epsilon_m \cdot \sigma \cdot T_{m,j}^3 \right)} \quad \text{seconds} \quad (2.32)$$

The solid-solid boundary nodes (nodes 6,8, and 10) have the following relationships;

$$\text{HEAT IN} = \left(\frac{k_{lm,j}}{\Delta x_l} \right) \cdot T_{l,j} + \left(\frac{k_{mn,j}}{\Delta x_n} \right) \cdot T_{n,j} \quad (2.33)$$

$$\text{HEAT OUT} = T_{m,j} \cdot \left[\frac{k_{lm,j}}{\Delta x_l} + \frac{k_{mn,j}}{\Delta x_n} \right] \quad (2.34)$$

$$\text{HEAT STORED} = \left[\frac{\rho_l \cdot CP_l \cdot \Delta x_l + \rho_n \cdot CP_n \cdot \Delta x_n}{2 \cdot \Delta t} \right] (T_{m,j+1} - T_{m,j}) \quad (2.35)$$

giving;

$$T_{m,j+1} = C_3 \left[\left(\frac{k_{lm,j}}{\Delta x_l} \right) \cdot T_{l,j} + \left(\frac{k_{mn,j}}{\Delta x_n} \right) \cdot T_{n,j} + T_{m,j} \cdot \left(\frac{1}{C_3} - \frac{k_{lm,j}}{\Delta x_l} - \frac{k_{mn,j}}{\Delta x_n} \right) \right] \quad (2.36)$$

$$C_3 = \frac{2 \cdot \Delta t}{\rho_l \cdot CP_l \cdot \Delta x_l + \rho_n \cdot CP_n \cdot \Delta x_n} \quad (2.37)$$

$$\Delta t \leq \frac{\rho_l \cdot CP_l \cdot \Delta x_l + \rho_n \cdot CP_n \cdot \Delta x_n}{2 \left(\frac{k_{lm,j}}{\Delta x_l} + \frac{k_{mn,j}}{\Delta x_n} \right)} \quad \text{seconds} \quad (2.38)$$

Finally, the interior surface node must satisfy the following relationships;

$$T_{m,j+1} = C_1 \cdot \left[\epsilon_m \cdot \sigma \cdot T_{int,j}^4 + h c_{int,j} \cdot T_{int,j} + \left(\frac{k_{lm,j}}{\Delta x_m} \right) \cdot T_{l,j} + T_{m,j} \cdot \left(\frac{1}{C_1} - \frac{k_{lm,j}}{\Delta x_m} - h c_{int,j} - \epsilon_m \cdot \sigma \cdot T_{m,j}^3 \right) \right] \quad (2.39)$$

$$C_1 = \frac{2 \cdot \Delta t}{\rho_m \cdot CP_m \cdot \Delta x_m} \quad (2.40)$$

$$\Delta t \leq \frac{\rho_m \cdot CP_m \cdot \Delta x_m}{2 \left(\frac{h_{i,m,j}}{\Delta x_m} + hc_{int,j} + \epsilon_m \cdot \sigma \cdot T_{m,j}^3 \right)} \quad \text{seconds} \quad (2.41)$$

Given the initial temperatures of all the nodes in the model (which can be determined using the steady state analysis if the transient analysis is started before dawn) and a suitably small time increment, the future temperature of each node can be calculated at each time interval. Because of the large number of temperature values this analysis can produce in the course of 24 hours, only hourly values will be saved.

Once the 24 sets of temperature values (one for each hour) have been determined, the set which gives the largest temperature differential between the frame and the veneer can quite easily be chosen.

2.3.1 Comparison of Predicted Transient Temperatures with Observed

As part of this study, the exterior wall of an apartment building in the Ottawa area was instrumented with temperature sensors. The sensors were placed on the outer surface and in the air cavity, as shown in Figure 2.9, at the top level of the south facing wall. Temperature measurements were made on an hourly basis.

The wall was made up of an exterior wythe of red clay brick veneer, 92 mm in width, a 25 mm air cavity, a 190 mm thick concrete blockwork inner wythe (loadbearing) with 25 mm of insulation followed by 12 mm of drywall on the inner surface. Beyond this data, nothing was known about the thermal characteristics of any of the wall materials, and typical values were assumed for use in the transient analysis, as seen in Table 2.1.

The windspeed was assumed to be 0.5 m/sec, the summer dawn temperature was 18 °C on July 27, 1982, and the maximum afternoon temperature was 30 °C. Based on this data, the transient analysis was run for a 24 hour period. The brick surface and air cavity predicted temperature values were extracted and plotted against the data observed on July 27, 1982 in Figures 2.10 and 2.11.

The transient heat flow analysis begins just prior to dawn with the initial temperatures predicted by the steady state thermal analysis. After a 24 hour period, the final temperatures are not expected to be precisely equal to the initial temperatures, which accounts for the discontinuity seen in the predicted temperature curve at 5:00 A.M.

The predicted brick surface temperature (Figure 2.10) shows very good agreement with that observed. Further adjustments could be made by 'fine tuning' the shading angles of the balconies,

wind speed, and the specific heat, emissivity, and absorptivity of the brick masonry. However, as is, the designer is given a good idea of the magnitude of error to be expected using some typical values.

Somewhat poorer agreement is observed between the predicted and observed air cavity temperatures seen in Figure 2.11. The predicted peak temperature occurs some four hours prior to the observed peak (although there is less than 2°C difference in their magnitudes). It must be remembered, however, that the air cavity temperatures were measured at the top of an air cavity which runs uninterrupted from the base of the building. It is felt that warm air rising from below, in particular from areas unshaded by balconies, results in the air cavity remaining warmer than predicted for a longer period of time. The predicted temperature variation is probably more representative of that occurring at the building midheight or in structures where the air cavity is interrupted by shelf angles.

Table 2.1 Material thermal characteristics used in the transient analysis example.

Material	Density (kg/m^3)	Conductivity ($\text{W}/\text{m}^{\circ}\text{C}$)	Specific Heat ($\text{kJ}/\text{kg}^{\circ}\text{C}$)	Emissivity	Absorptivity
Brick	2300	1.950	0.90	0.90	0.80
Block	2000	1.320	1.03	0.90	0.57
Ins.	24	0.043	1.21	0.90	0.60
Finish	960	0.136	1.09	0.90	0.60

Figure 2.9 Roof level instrumentation to measure surface and air cavity temperatures.

Figure 2.10 Observed and predicted brick surface temperatures over 24 hours.

Figure 2.11 Observed and predicted air cavity temperatures over 24 hours.

Chapter 3

Prediction of Movement

3.1 Introduction

All materials will experience dimensional changes when subjected to certain conditions. The required conditions may vary from one material to the next, but in all cases the magnitude of the movement can be predicted if the deformational properties of the material are known. For instance, if steel is subjected to a load or a temperature variation, its linear dimensions will change. The amount of change can easily be estimated if the elastic modulus and coefficient of thermal expansion of the steel is known.

While all materials exhibit elastic strains and most exhibit thermal strains, some experience more complex movements which vary with time, such as creep, shrinkage, and moisture expansion. Although these movements are generally dependent on a wider variety of material properties, they too can be estimated if the properties are known.

In the following sections, the deformational properties of the most common building materials, namely concrete, concrete blockwork, clay brick masonry, and steel, will be discussed. The proposed facts and relationships allow the estimation of the unrestrained movements in each material when subjected to conditions encountered in a given structure.

3.2 Concrete

Next to metals, the properties of concrete, as a building material, are probably the most well known. Because of its high strength and low cost, concrete is popular throughout the world and, as a result, considerable time and effort has been spent in determining its structural and mechanical characteristics. The deformational properties of concrete, however, are the only characteristics of interest in this report.

Like most structural materials, concrete experiences reversible movements due to thermal and elastic strains, the magnitudes of which will be discussed in the following sections. Unlike steel, however, concrete also undergoes gradual irreversible volume changes due to drying shrinkage and creep. These will also be discussed in greater detail in the following sections.

Because concrete is usually reinforced, the deformations will be restrained by the enclosed steel reinforcement. To determine the 'net' movement of a reinforced concrete column it is necessary to find the unrestrained deformations of both the concrete and the steel, and combine them using

compatibility and elastic relationships. For this reason, the magnitudes of the deformations predicted in the following sections will be unrestrained values (ie, as if the concrete is unreinforced). The deformational properties of steel are discussed in Section 3.5

3.2.1 Creep of Concrete

Although a great number of relationships between the creep of concrete, applied stress, and time have been proposed over the years, very few of them consider the effects of incremental loading (ie, most consider stress to be constant over the period of interest). A column at a lower level of a tall building will experience a gradual increase in stress over the construction period which may be up to a year in duration. Since the amount of creep in concrete, due to a unit stress, decreases with the age of the concrete at the time the stress was applied, the creep in a low level column will be considerably less than that predicted by most creep relationships.

In order to determine the creep due to incremental loading it is necessary to apply the principle of superposition, which states that, within the range of service stresses, the creep deformations due to stress fractions applied at different intervals of time are considered to be additive (C.E.B., 24). This in turn implies that for stresses less than 0.5 to 0.6 of the ultimate strength of the concrete, the magnitude of the creep is directly related to the magnitude of the applied stress. This has been found to be true for concrete.

Of the methods proposed to predict the creep strain due to incremental loading, the best (for the purposes of this report) was found to be that put forward by the Comité Européen de Béton (C.E.B., 23). Both Fintel et al. (35), and Grimm (40) present simplified methods but both have the following disadvantages,

- 1) The creep strain during the construction phase is difficult to determine
- 2) Since they rely on the final stress (after completion of the building) on the concrete column, the effects of transfer of load to the blockwork or brick masonry cannot be considered (since the magnitude of this transfer depends on the magnitude of the deformation in the column)

Prediction of Magnitude of Creep

The creep strain in concrete at any instant in time due to a stepwise varying stress is proposed by C.E.B. (23) to be,

$$\epsilon_{crCL}(t_i) = \sum_{j=1}^{(i-1)} k_j (\sigma_j - \sigma_{j-1}) \phi(t_i, t_j) \quad (3.1)$$

where,

ϵ_{crCL} = total unrestrained creep strain in the concrete at the time of interest t_i (at the end of interval $i - 1$)

σ_j = total stress applied to the concrete at the beginning of interval j in MPa (assumed to be constant throughout the interval) - compression is negative

k_j = creep recovery correction factor

= 1.0 if $(\sigma_j - \sigma_{j-1}) \leq 0$ (ie. compression force increasing)

= 0.67 if $(\sigma_j - \sigma_{j-1}) > 0$ (ie. compression force decreasing)

$\Phi(t_i, t_j)$ = creep function giving the specific creep at the time of interest t_i due to the stress increment applied at the beginning of interval j (t_j) in mm/mm/MPa

t_i = age of the concrete at the time of interest (days)

t_j = age of the concrete at the beginning of interval j (days)

The creep recovery correction factor is based on the observation made by Neville (77) that creep recovery (due to unloading) is only about 2/3 that predicted by the principle of superposition. The creep function $\Phi(t_i, t_j)$ is given by C E B. to be,

$$\Phi(t_i, t_j) = \frac{1}{EC(28)} [\beta_a(t_i) + \psi_d \beta_d(t_i - t_j) + \psi_{f1} \psi_{f2} \{\beta_f(t_i) - \beta_f(t_j)\}] \quad (3.2)$$

where,

$EC(28)$ = mean 28 day modulus of elasticity of concrete (MPa)

= $11875 \sqrt{f'_c(28)}$ (MPa) from C E B. (23) (3.3)

$f'_c(28)$ = 28 day concrete cylinder compressive strength (MPa)

$\beta_a(t_j)$ = plastic flow developed in the first day after load application

= $0.8 \left(1 - \frac{f'_c(t_j)}{f'_c(\infty)} \right)$ (see Figure 3.1)

= $0.8 - 0.16948 \ln(1.16306 t_j) + 0.16785 \ln \left(1 + \frac{t_j}{93.09} \right)$ (3.4)

ψ_d = coefficient for delayed elastic strain

= 0.4

$\beta_d(t_i - t_j)$ = delayed elastic strain function (see Figure 3.2)

= $0.17196 \ln(0.57236(t_i - t_j)) + 0.07541 \ln \left(1 + \left(\frac{16.5576}{t_i - t_j} \right)^{1.8} \right)$
 $- 0.08855 \ln \left(1 + \left(\frac{t_i - t_j}{637.71} \right)^2 \right)$ (3.5)

$$\begin{aligned}\psi_{f_1} &= \text{relative humidity flow coefficient (see Figure 3.3)} \\ &= 4.1034 \left(1 - \frac{R.H. \%}{100} \right)^{0.613}\end{aligned}\quad (3.6)$$

$R.H. \% =$ relative humidity of the environment (%)

$$\begin{aligned}\psi_{f_2} &= \text{effective thickness flow coefficient (see Figure 3.4)} \\ &= 5.67525 \left(\frac{1}{d_m} \right)^{0.05} - 2.8045\end{aligned}\quad (3.7)$$

$$\begin{aligned}d_m &= \text{effective thickness of the column (mm)} \\ &= \lambda \left(\frac{2A_c}{P} \right)\end{aligned}\quad (3.8)$$

$$\begin{aligned}\lambda &= \text{ambient environment coefficient (see Figure 3.3)} \\ &= 0.98 + 4.0478 \times 10^{-4} e^{0.10226(R.H. \%)}\end{aligned}\quad (3.9)$$

$A_c =$ cross-sectional area of the column (mm^2)

$P =$ perimeter length of the column cross-section (mm)

$\beta_f(t) =$ delayed plastic flow function (see Figure 3.5)

The delayed plastic flow function β_f is quite complex, varying both with time and the effective thickness d_m . For hand calculations the designer is better off to go directly to the curves presented in Figure 3.5 and interpolate for his particular value of d_m . The following relationship has been developed for the computer to allow a fairly precise evaluation of β_f ;

$$\beta_f(t) = a \ln(bt) + c \ln \left(1 + \left(\frac{d}{t} \right)^P \right) - f \ln \left(1 + \left(\frac{t}{g} \right)^Q \right) \quad (3.10)$$

where the coefficients a to Q vary for different values of d_m as shown in Table 3.1. To interpolate for intermediate values of d_m (for example $d_m = 275 \text{ mm}$);

- 1) Let d_1 be the next lower value of d_m for which an equation or curve exists (ie. $d_1 = 200 \text{ mm}$), and d_2 be the next higher (ie. $d_2 = 400 \text{ mm}$).
- 2) Calculate $\beta_{f_1}(t)$ for d_1 (ie. for $d_m = 200$) and $\beta_{f_2}(t)$ for d_2 (ie. for $d_m = 400$).
- 3) Calculate the interpolation factor α to be;

$$\alpha = \left[\frac{d_m - d_1}{d_2 - d_1} \right]^{0.6} = \left[\frac{275 - 200}{400 - 200} \right]^{0.6} \quad (3.11)$$

- 4) Calculate the desired $\beta_f(t)$ value as follows;

$$\beta_f(t) = \beta_{f_1}(t) - \alpha [\beta_{f_1}(t) - \beta_{f_2}(t)] \quad (3.12)$$

Summary of Creep Strain Relationships

$$\epsilon_{crCL}(t_i) = \sum_{j=1}^{(i-1)} k_j \cdot (\sigma_j - \sigma_{j-1}) \cdot \Phi(t_i, t_j) \quad (3.1)$$

where,

$$k_j = 1.0 \quad \text{if } (\sigma_j - \sigma_{j-1}) \leq 0 \\ = 0.67 \quad \text{if } (\sigma_j - \sigma_{j-1}) > 0$$

$$\Phi(t_i, t_j) = \frac{1}{EC(28)} [\beta_a(t_j) + 0.4\beta_d(t_i - t_j) + \psi_{f_1} \cdot \psi_{f_2} \cdot \{\beta_f(t_i) - \beta_f(t_j)\}] \quad (3.2)$$

$$EC(28) = 11875 \sqrt{f'_c(28)} \quad (3.3)$$

$$\beta_a(t_j) = 0.8 - 0.16948 \ln(1.16306 t_j) + 0.16785 \ln \left(1 + \frac{t_j}{93.09} \right) \quad (3.4)$$

$$\beta_d(t_i - t_j) = 0.17196 \ln(0.57236(t_i - t_j)) + 0.07541 \ln \left(1 + \left(\frac{16.5576}{t_i - t_j} \right)^{1.8} \right) \\ - 0.08855 \ln \left(1 + \left(\frac{t_i - t_j}{637.71} \right)^2 \right) \quad (3.5)$$

$$\psi_{f_1} = 4.1034 \left(1 - \frac{R.H.\%}{100} \right)^{0.613} \quad (3.6)$$

$$\psi_{f_2} = 5.67525 \left(\frac{1}{d_m} \right)^{0.05} - 2.8045 \quad (3.7)$$

$$d_m = \lambda \cdot \frac{2A_c}{P} \quad (3.8)$$

$$\lambda = 0.98 + 4.0478 \times 10^{-4} e^{0.10226(R.H.\%)} \quad (3.9)$$

$$\beta_f(t) = a \ln(bt) + c \ln \left(1 + \left(\frac{d}{t} \right)^P \right) - f \ln \left(1 + \left(\frac{t}{g} \right)^Q \right) \quad (3.10)$$

(see Table 3.1)

$$\beta_s = 0.8 \left(1 - \frac{f'_s(t_j)}{f'_s(\infty)} \right)$$

Figure 3.1 Variation of the first day plastic flow function β_s with age at loading t_j .

Figure 3.2 Variation of the delayed elastic strain function $\beta_d(t, -t_j)$ with load duration $(t, -t_j)$

Table 3.1 Coefficients for use in Equation (3.10) for the calculation of the delayed plastic flow in concrete.

d_m	a	b	c	d	f	g	P	Q
50	0.16603	0.41521	0.06664	6.1055	0.10163	749.792	1.50	1.50
100	0.16036	0.40458	0.04326	9.8208	0.10755	1141.435	2.00	1.40
200	0.16026	0.31470	0.06444	11.5717	0.14694	1769.201	1.50	1.10
400	0.14482	0.23919	0.11935	13.1296	0.11154	5415.922	0.80	1.45
800	0.14913	0.11974	0.08257	86.8109	0.07507	7666.979	1.00	2.10
1600	0.19089	0.02592	0.07348	290.8034	0.11871	8979.504	1.80	1.80

Figure 3.3 Relative humidity flow coefficient ψ_{f1} and the ambient environment coefficient λ variation with relative humidity.

Figure 3.4 Effective thickness flow coefficient ψ_{fs} .

Figure 3.5 Variation of the delayed plastic flow function β_f with time under load for various values of effective thickness d_m

3.2.2 Shrinkage of Concrete

The gradual moisture loss from concrete results in a slight volume change which we call shrinkage. The amount of shrinkage an element will experience depends on;

- 1) The age of the concrete.
- 2) The relative humidity of the environment.
- 3) The dimensions of the element
- 4) The composition of the concrete
- 5) The temperature of the environment

The last factor only affects the rate of development of the shrinkage

Grimm (40) gives a relationship for the relative shrinkage strain taking place in concrete between the times t_e and t_i ,

$$\epsilon_{sh_{e,i}}(t_i, t_e) = \epsilon_{sh_t} \left[N \frac{t_i}{K_c + t_i} - N \frac{t_e}{K_c + t_e} \right] \quad (3.13)$$

where,

$\epsilon_{sh_{e,i}}$ = concrete shrinkage strain occurring between the times t_e and t_i

t_i = age of the concrete at the time of interest (days)

t_e = age of the concrete at the earlier time of interest (days)
(for example, when the shelf angle was placed)

K_c = curing condition coefficient

= 1.0 for 7 day moist cure

= 1.57 for 1 to 2 day steam cure

$$N = 26e^{1.18 \times 10^{-2}(V/S)} \quad (3.14)$$

V/S = volume to surface ratio of the element in question (mm)

ϵ_{sh_t} = ultimate concrete shrinkage strain at 23 years

The ultimate concrete shrinkage strain at 23 years is given by Grimm to be;

$$\epsilon_{sh_t} = \epsilon_{at} \cdot (1 - 2.35 \times 10^{-7}(R.H.\%)^3) (0.89 + 0.0016 \cdot S)(0.75 + 6.1 \times 10^{-4}\beta_c) \\ \times (0.3407 \ln(Fa) - 0.3786)(0.95 + 0.008a)(1 + 0.05c) \quad (3.15)$$

where,

ϵ_{at} = unadjusted ultimate concrete shrinkage at 23 years as a function
of aggregate type (see Table 3.2)

$R.H.\%$ = relative humidity of the environment (%)
if $R.H.\% < 40\%$ use $R.H.\% = 40\%$

S = slump of concrete (mm)

β_c = cement content (kg/m^3)

F_a = ratio of concrete fine aggregate to total aggregate content by weight

a = air content of concrete (%), for $a < 6$ use $a = 6\%$

c = ratio of calcium chloride to cement content in concrete by weight (%)

C.E.B. (23) predicts the shrinkage strain in concrete to be;

$$\epsilon_{sh_{CL}}(t_i, t_e) = \epsilon_{s_0} \cdot [\beta_S(t_i) - \beta_S(t_e)] \quad (3.16)$$

where,

$$\begin{aligned} \epsilon_{s_0} &= \text{ultimate shrinkage strain} \\ &= \epsilon_{s_1} \cdot \epsilon_{s_2} \end{aligned} \quad (3.17)$$

$$\begin{aligned} \epsilon_{s_1} &= \text{unadjusted ultimate shrinkage strain of concrete} \\ &= 7.7211 \times 10^{-4} \left(1 - \frac{R.H.\%}{100} \right)^{0.7737} \quad (\text{see Figure 3.6}) \end{aligned} \quad (3.18)$$

$$\begin{aligned} \epsilon_{s_2} &= \text{correction factor for effective thickness } (d_m) \\ &= 0.6 + \left(\frac{18}{d_m} \right)^{0.5} \quad (\text{see Figure 3.7}) \end{aligned} \quad (3.19)$$

$\beta_S(t)$ = variation of shrinkage with time function (see Figure 3.8)

The unadjusted ultimate shrinkage strain value (ϵ_{s_1}) tends to be somewhat lower than the typical values observed in North America. For example, C.E.B. predicts the ultimate shrinkage strain at 50% relative humidity to be;

$$\epsilon_{s_1} = 451 \times 10^{-6} \text{ mm/mm}$$

Grimm, on the other hand, presents a range of values for shrinkage at 50% relative humidity from 550×10^{-6} to 1260×10^{-6} mm/mm which is in agreement with Neville (78), Hobbs (46), and ACI (3). However, Baker (6), BRE (17), Sahlin (93), and Winter (117) all give shrinkage values ranging between 200 and 800×10^{-6} mm/mm. Thus, although the C.E.B. relationship may be somewhat conservative, it was adopted for use in the computer program to calculate the ultimate shrinkage of concrete if a known value is not specified by the user.

The variation of shrinkage with time, $\beta_S(t)$, depends on the effective thickness of the concrete element (ignoring temperature fluctuations) and was derived from the C.E.B. observations as follows;

$$\beta_S(t) = h \cdot \ln(j \cdot t) + k \cdot \ln \left(1 + \left(\frac{m}{t} \right)^P \right) - n \cdot \ln \left(1 + \left(\frac{t}{r} \right)^Q \right) \quad (3.20)$$

where the coefficients h through r depend on the effective thickness d_m and are shown in Table 3.3.

To interpolate for intermediate values of d_m (for example, $d_m = 275$ mm) the following method should be used;

- 1) Let d_1 be the next lower value of effective thickness shown in Table 3.3 (ie. for $d_m = 275$ mm, $d_1 = 200$ mm) and d_2 be the next higher value (ie. $d_2 = 400$ mm).
- 2) Calculate $\beta_{S_1}(t)$ for the effective thickness d_1 and $\beta_{S_2}(t)$ for d_2 .
- 3) Calculate an interpolation factor α as follows;

$$\alpha = [1.2 - 0.0869 \ln(t)] \left[\frac{d_m - d_1}{d_2 - d_1} \right]^{(1.8/d_m)^{0.7}} \quad (3.21)$$

- 4) Calculate $\beta_S(t)$ as follows;

$$\beta_S(t) = \beta_{S_1}(t) - \alpha \cdot [\beta_{S_1}(t) - \beta_{S_2}(t)] \quad (3.22)$$

Table 3.2 Ultimate shrinkage strains in various concrete types for use in Equation (3.15) (from Grimm, 40).

Aggregate Type	ϵ_{aU} $\mu m/m$
Quartz	550
Limestone	650
Basalt	850
Granite	850
Gravel	1140
Sandstone	1260
Lightweight	
low	800
mean	1150
high	1170

Table 3.3 Coefficients for use in the calculation of the shrinkage function $\beta_S(t)$ according to Equation (3.20).

d_m	h	j	k	m	n	r	P	Q
50	0.19464	0.58397	0.88267	5.1596	0.13073	283.108	1.70	1.45
100	0.18801	0.27228	0.08482	7.6606	0.10532	662.446	1.75	1.70
200	0.21292	0.06315	0.17134	22.8553	0.06661	1355.882	1.10	2.80
400	0.24861	0.01728	0.14650	98.8952	0.08884	2822.085	1.50	2.60
800	0.36080	0.00312	0.18806	468.9953	0.10275	4683.308	1.80	3.30
1600	0.36503	0.00150	0.16160	793.5471	0.11509	9451.367	2.20	3.00

Figure 3.6 Variation in the unadjusted ultimate shrinkage strain of concrete (ϵ_{s1}) with ambient relative humidity.

Figure 3.7 Variation of the shrinkage correction factor ϵ_s with effective thickness d_m

Figure 3.8 Variation of shrinkage function $\beta_s(t)$ with time for various values of effective thickness d_m

3.2.3 Elasticity of Concrete

The modulus of elasticity of concrete has been related to a number of parameters, the most popular being density and compressive strength. The CSA code A23.2-M77 (18) gives the modulus of elasticity of concrete as;

$$EC = 0.043w^{1.5}\sqrt{f'_c} \quad (\text{MPa}) \quad (3.23)$$

$$\simeq 5000\sqrt{f'_c} \quad \text{for normal density concrete}$$

where,

w = density of concrete (kg/m^3)

f'_c = specified compressive strength of concrete (MPa)

The value f'_c is a lower bound on the true strength of a given concrete mix and as such the elastic moduli given by the CSA code tends to overestimate the elastic strain experienced by concrete under a given load. To avoid this problem, it may be better to use the relationship proposed by C.E.B. (24) for the mean modulus of elasticity of normal weight concrete at 28 days; (see Fig. 3.9)

$$EC(28) = 11875 \sqrt[3]{f'_{cm}(28)} \quad (\text{MPa}) \quad (3.24)$$

where,

$f'_{cm}(28)$ = 28 day mean compressive strength of concrete based on cylinder tests (MPa)

The variation of the elastic modulus of concrete with time was found by C.E.B. to be;

$$EC(t_i) = EC(28) \sqrt[3]{\frac{f'_{cm}(t_i)}{f'_{cm}(28)}}$$

$$= EC(28) \cdot \left[0.31029 \ln(1.16306t_i) - 0.30730 \ln \left(1 + \frac{t_i}{93.09} \right) \right]^{\frac{1}{3}} \quad (3.25)$$

where t_i is the age of the concrete at the time of interest in days. Grimm (40) proposes a simpler relationship;

$$EC(t_i) = EC(28) \sqrt{\frac{t_i}{4 + 0.85t_i}} \quad (3.26)$$

Figure 3.9 Variation of concrete elastic modulus with compressive strength for normal weight concrete (data from C.F.B., 24)

3.2.4 Thermal Strain in Concrete

The coefficient of thermal expansion of concrete depends on the characteristics of its constituent materials (ie. cement paste, fine, and coarse aggregate) as well as the concrete's age and moisture content. Of these, the most difficult to estimate is the moisture content. In most cases, if the moisture content is defined as the percentage of total absorption, it can be estimated to be about equal to the relative humidity of the environment.

Emanuel and Hulsey (32) estimate the coefficient of thermal expansion of concrete to be;

$$\alpha_{co} = f_t \cdot [(\beta_p \cdot \alpha_p) + (\beta_{fa} \cdot \alpha_{fa}) + (\beta_{ca} \cdot \alpha_{ca})] \quad (3.27)$$

where,

α_{co} = linear coefficient of thermal expansion of concrete (per °C)

α_p = coefficient of thermal expansion of cement paste
(see Table 3.4)

α_{fa} = coefficient of thermal expansion of the fine aggregate
(see Table 3.5)

α_{ca} = coefficient of thermal expansion of the coarse aggregate
(see Table 3.5)

β_p = proportion by volume of cement paste
= 0.25 if unknown

β_{fa} = proportion by volume of fine aggregate
= 0.225 if unknown

β_{ca} = proportion by volume of coarse aggregate
= 0.525 if unknown

f_t = correction factor for temperature fluctuations
= 1.0 for a controlled environment
= 0.86 for an exposed environment

Note that,

$$\beta_p + \beta_{fa} + \beta_{ca} = 1.0 \quad (3.28)$$

Table 3.4 Thermal coefficient of expansion of cement paste in $\mu m/m^{\circ}C$ (from Emanuel et al, 32).

Moisture Content (m_c) (%)	Age of Concrete		
	0-6 Months	12-18 Months	> 15 Years
0-20	12.6	12.6	12.6
20-40	$9.0 + 0.18m_c$	$10.0 + 0.13m_c$	$9.3 + 0.16m_c$
40-60	$9.0 + 0.18m_c$	$10.0 + 0.13m_c$	15.9
60-75	19.8	17.8	$26.0 - 0.17m_c$
75-90	$64.8 - 0.6m_c$	$52.8 - 0.47m_c$	$26.0 - 0.17$
90-100	10.8	10.8	10.8

Table 3.5 Average coefficients of linear thermal expansion for rocks over normal temperature ranges (from Emanuel et al, 32).

Type of rock or mineral	Average coefficient in $\mu m/m^{\circ}C$
Quartzite, silica shale, cherts	11.0-12.5
Sandstones	10.5-12.0
Quartz, sands, pebbles	10.0-12.5
Clay, mica shales	9.5-11.0
Granites, gneisses	6.5-8.5
Syenites, feldspathic porphyry diorites, andesite, phonolite gabbros, diabase, basalt	5.5-8.0
Dense, crystalline, porous limestones	3.5-6.0
Pure calcite	4.0-6.5
Marbles	4.0-7.0
Dolomites, magnesites	7.0-10.0

Grimm (40) gives a simplified relationship for the coefficient of thermal expansion of concrete,

$$\alpha_{cs} = C_c + 1.26 \times 10^{-7} \left[4.3 - \frac{A}{C} \right] \quad (3.29)$$

where,

C_c = coefficient of thermal expansion of concrete (per °C)

(see Table 3.6)

A/C = aggregate to cement weight ratio

The second parameter has little effect on α_{cs} and so if A/C is not known set $\alpha_{cs} = C_c$.

The unrestrained thermal strain in the concrete at the time of interest can now be calculated to be;

$$\epsilon_{th_{CL}} = \alpha_{cs} \cdot (T_{i_{CL}} - T_{e_{CL}}) \quad (3.30)$$

where,

$\epsilon_{th_{CL}}$ = unrestrained thermal strain in the concrete

$T_{i_{CL}}$ = mean temperature of the column (CL) at the time of interest in °C

$T_{e_{CL}}$ = mean temperature of the column at the earlier time of interest in °C
(ie. when placed or when masonry was placed)

Table 3.6 Linear coefficients of thermal expansion of concrete for different aggregate types (from Grimm,40).

Aggregate Type	Thermal Coefficient ($\mu m/m^\circ C$)
Quartz	11.9
Limestone	6.8
Basalt	8.6
Granite	9.5
Gravel	10.8
Sandstone	11.7
Lightweight	8.1

3.3 Clay Brick Masonry

3.3.1 Introduction

When clay brick masonry first became popular as a facade on buildings it was assumed to have deformational properties similar to those of concrete. It was later realized that brick masonry often undergoes expansion with time. This phenomenon is due to moisture absorption by the brick, and considerable research has since allowed the magnitude of moisture expansion to be estimated.

Clay brick masonry also experiences elastic shortening, creep, thermal movements, and of particular interest in Canada, freezing expansion. These deformational properties will be discussed in greater detail in the following sections.

3.3.2 Creep of Clay Brick Masonry

The creep of clay brick masonry has been found by several researchers (40,55,63,66,86,115) to be proportional to stress, provided that the stress does not exceed approximately 1/2 the ultimate strength of the brick masonry. Therefore the creep due to a certain incremental stress $\Delta\sigma_1$, plus the creep due to a different incremental stress $\Delta\sigma_2$, is just equal to the creep experienced by the brick masonry under the stress $(\Delta\sigma_1 + \Delta\sigma_2)$ (principle of superposition).

The principle of superposition greatly simplifies the determination of creep under incremental loading. The total creep at a particular time can be calculated as the sum of the incremental creep values due to the incremental stresses. To accomplish this, a relationship between creep and time, as well as age at loading is required.

Although a fair amount of research has been done on the relationship between creep and time, very little has been done on the effect of the age at loading. Grimm (40) assumes that masonry is similar to concrete in this respect and calculates the total creep directly using the following relationship,

$$\epsilon_{crBR} = \frac{\sigma_t}{E_{mx}} \left[(2595 \times 10^{-6})(t_l)^{-0.118} (189 - R.H.\%) (5.1 + a_m) (\ln(f'_{mx}) - 1.12) - K_t \right] \quad (3.31)$$

where,

ϵ_{crBR} = creep strain in the masonry

σ_t = axial stress acting on the masonry at the time of interest (MPa)

E_{mx} = tested modulus of elasticity (MPa) at the time x

t_l = age of the masonry at the time of loading (days)

R.H.% = relative humidity of mortar drying air (%)

a_m = air content of the mortar (%)

f'_{mx} = tested compressive strength of the masonry (MPa) at the time x

x = age of the masonry at the time of testing (days)

K_t = a weighted time function to account for increasing stress over the period from time of loading to the time of interest

To determine the weighted time function, K_t , Grimm assumes that the specific creep (C_{st}) under constant stress in masonry relates to time as it does for concrete;

$$C_{st} \propto \frac{t^{0.6}}{10 + t^{0.6}} \quad (3.32)$$

For stress increasing linearly over the period considered (from time of loading, t_l , to the time of interest, t_i), Grimm expects that;

$$C_{st} \propto \frac{1}{t_i - t_l} \int_{t_l}^{t_i} \left(\frac{t^{0.6}}{10 + t^{0.6}} \right) dt \quad (3.33)$$

To include the effect of changing stiffness, Grimm includes the term;

$$g(t) = \ln \left[f'_{ms} \left(\frac{31 + \ln(t_i)}{31 + \ln(t_l)} \right) - 1.12 \right] \quad (3.34)$$

which gives, finally;

$$K_t = \frac{1}{t_i - t_l} \int_{t_l}^{t_i} \left(\frac{t^{0.6}}{10 + t^{0.6}} \right) \left(\frac{1}{g(t)} \right) dt \quad (3.35)$$

Grimm's age at loading correction factor ($t_l^{-0.118}$) in Eq. (3.31) should actually appear inside the integration in Equation (3.35), as it does in the C.E.B. relationships, modifying the specific creep as the stress changes with time.

If Grimm's age at loading correction factor is normalized to be approximately 1.0 at a loading age of 28 days, it can be compared to the age at loading functions of both Poljakov (86), to be discussed next, and C.E.B. (presented by Neville, 77), as seen in Table 3.7

Note that Grimm assumes that the rate of creep, if loaded beyond 28 days, is substantially larger than either Poljakov or C.E.B. predict it to be. The age at loading correction factor cannot be obtained directly from the latest C.E.B. relationships used in Section 3.2.1, and so the values shown in Table 3.7 were taken from an older C.E.B. relationship presented by Neville (77).

Poljakov (86) takes a much closer look at the effect of the age at loading. He arrives at the relationship;

$$\epsilon_{crss} = A \left(\frac{\sigma_i}{\sigma_{ult}} \right) \left(0.1 + 1.82 \epsilon^{-0.3} \sqrt{t_i} \right) (t_i - t_l)^{\frac{1}{2}} \quad (3.38)$$

where,

ϵ_{crss} = creep strain in the masonry

A = a constant dependent on the brick type

Table 3.7 Comparison of the age at loading correction factors derived from the sources indicated.

Age at Loading (days)	Correction Factor		
	Grimm	Poljakov	C.E.B.
7	1.19	1.22	1.37
14	1.10	1.12	1.17
28	1.01	1.01	1.00
50	0.95	0.92	0.83
100	0.87	0.81	0.68
200	0.80	0.69	0.55
500	0.72	0.54	0.38
1000	0.66	0.44	0.27
3000	0.58	0.30	0.20
7000	0.53	0.22	0.15

σ_i = the applied stress (MPa)

σ_{ult} = average ultimate strength of the brick masonry (MPa)

t_i = age of the brick masonry at the time of loading (days)

t_r = age of the brick masonry at the time of interest (days)

Poljakov's relationship shows relatively poor agreement with Lenczner's work (63,66,115). It is believed that this is due to a lack of understanding as to how σ_{ult} is defined in the Russian system. The correction Poljakov makes for the age at loading is given by the term $(0.1 + 1.82e^{-0.3} \sqrt[3]{t_i})$ which is shown for various values of t_i in Table 3.7. Since a large proportion of the creep is believed to take place in the mortar joint (13), the age at loading correction factor would be expected to approach that found for concrete. In this respect the correction factor given by Poljakov appears to be quite reasonable.

Lenczner (63,66,115) has conducted an intensive study into the relationship between creep and time for walls loaded at 28 days. The following relationships have been derived from the work done by Warren and Lenczner (115).

$$\epsilon_{creep} = \sigma_i \times 10^{-5} \left[\frac{(t_i - 28)}{a + b(t_i - 28)} \right] \quad (3.39)$$

where the constants a and b are determined from the brick unit strength (f'_{br}) and construction conditions as follows;

1) For bricks laid dry,

$$b = \frac{5.171\sqrt{f'_{br}} - 19.158}{734.6 - 61.53\sqrt{f'_{br}}} \quad (3.40)$$

$$a = 3.8024 \ln(b) + 18.2096 \quad (3.41)$$

Equations (3.40) and (3.41) are considered reliable for values of f'_{br} between 22 and 143 MPa.

2) For bricks laid wet,

$$b = \frac{5.171\sqrt{f'_{br}} - 19.158}{325.4 - 30.58\sqrt{f'_{br}}} \quad (3.42)$$

$$a = 7.3876 \ln(b) + 21.7915 \quad (3.43)$$

Equations (3.42) and (3.43) are considered reliable for values of f'_{br} between 29 and 113 MPa.

It is assumed that the difference between the wet and dry relationships reflects the effect of brick suction on the mortar properties. Since North American bricks in general have lower rates of suction than their British counterparts, it is believed that the 'wet' relationships more closely describe the North American brick masonry, even when bricks are laid dry.

The major drawback to Lenczner's work is that it contains no adjustment factor to take into account the age of the brick masonry when loaded (if different than 28 days). If the final creep function takes a multiplicative form, i.e.;

$$\epsilon_{crBR}(t_i, t_l) = \epsilon_{crBR}(t_i, 28) \times \{\text{correction factor}\} \quad (3.44)$$

then Poljakov's 'age at loading' relationship can be used as the correction factor (note that its value is approximately 1.0 at 28 days), giving;

$$\epsilon_{crBR}(t_i, t_l) = (\sigma_l \times 10^{-5}) \left[\frac{(t_i - t_l)}{a + b(t_i - t_l)} \right] \left(0.1 + 1.82e^{-0.3 \sqrt[3]{t_l}} \right) \quad (3.45)$$

where σ_l is now the stress applied at the time of loading (t_l) in MPa. Now the creep in brick masonry loaded at, say, 50 days can be determined by calculating the creep as if loaded at 28 days and adjusting by the age at loading factor.

Equation (3.45) can be applied to the case of incremental loading by using the principle of superposition;

$$\epsilon_{crBR}(t_i) = \sum_{j=1}^{(i-1)} \left\{ (k_j)(\Delta\sigma_j \times 10^{-5}) \left[\frac{(t_i - t_j)}{a + b(t_i - t_j)} \right] \left(0.1 + 1.82e^{-0.3 \sqrt[3]{t_j}} \right) \right\} \quad (3.46)$$

where,

$$\begin{aligned}
 k_j &= \text{correction factor to provide for the overestimation of creep recovery by the principle of superposition} \\
 &= 1.0 \quad \text{if } \Delta\sigma_j \geq 0 \\
 &= 0.67 \quad \text{if } \Delta\sigma_j \leq 0
 \end{aligned}$$

$$\begin{aligned}
 \Delta\sigma_j &= \text{incremental stress applied at the time } t_j \text{ (MPa)} \\
 &= \sigma_j - \sigma_{j-1}
 \end{aligned}$$

$$t_j = \text{age of the masonry at the beginning of interval } j \text{ (days)}$$

3.3.3 Moisture Expansion of Clay Brick Masonry

Immediately after firing, clay brick begins to absorb moisture from the atmosphere, which in turn participates in complex chemical reactions within the clay itself. The absorption process will continue until an equilibrium state with the environment is attained. This phenomena is of interest to the designer because it results in a gradual growth or expansion of the clay, the rate of expansion being directly proportional to the rate of moisture absorption. Both the rate and the magnitude of the clay brick expansion are effected by such parameters as the temperature the brick was fired at, the humidity and temperature of its final environment, and the chemical composition of the clay. These parameters will be discussed in greater detail in this section.

Time of Exposure

Most researchers are in agreement that the amount of moisture expansion varies with the logarithm of time. Both Grimm (40) and McDowall and Birtwistle (69) present the relationship,

$$\epsilon_{bm} \propto \ln(t + 2.3) \quad (3.47)$$

where,

$$\begin{aligned}
 t &= \text{age of the brick since 24 hours after removal from kiln (months)} \\
 \epsilon_{bm} &= \text{brick unit moisture expansion strain}
 \end{aligned}$$

which gives reasonably good agreement with the findings of Beard et al (8), Laird et al (59), as well as Hosking et al (48,49). It is significantly in error however for Smith's (102) findings when the kiln to wall time lapse is greater than 1 year. It appears that the logarithmic relationship predicts too rapid a growth during the early period, but the results are quite reasonable for kiln to wall time lapses of less than about 3 months.

Temperature of Environment

Although very little research has been done on the effect of the environmental temperature on moisture expansion, the findings of Beard et al (8) indicate that south facing brick masonry will expand by up to 30% more than the shaded north facing leaves.

Temperature of Firing

Expansions rise to a maximum for brick fired between 900 °C and 1050 °C, thereafter decreasing. These temperatures correspond to the lower end of the industrial firing range and, therefore, most bricks produced show a decrease in expansion with increase in firing temperature (58).

Humidity of the Environment

Both the rate and the magnitude of moisture expansion of clay brick depends on the relative humidity of the air in which the brick is stored. According to Ritchie (90), a considerable difference in the rate of moisture expansion was observed for different relative humidities of the drying air (the higher the relative humidity, the faster the initial rate of expansion). Ritchie's observations are presented in Figures 3.10 and 3.11.

Chemical Composition

Moisture expansion is due largely to chemical reactions taking place within the clay. Because of this, the amount and rate of expansion changes significantly with changes in the chemical composition of the clay. Therefore, the great body of research into moisture expansion which has taken place outside of Canada must be used with some caution for bricks produced within Canada.

Figure 3.10 Expansion of clay and shale bricks (from Ritchie, 90).

Figure 3.11 Effect of humidity on brick expansion (from Ritchie, 90).

Estimation of Magnitude of Permanent Moisture Expansion

Probably the best methods of predicting moisture expansion found were those proposed by McDowall et al (69) and Grimm (40). McDowall's approach is based on a 4 hour steam expansion test which they say shows reasonable agreement with expansion under natural exposure for any time less than about 4 years,

$$\epsilon_{bm} = \epsilon_{bmt} \{0.6013 \ln(t + 2.298) - 0.1929\} \quad (3.48)$$

where ϵ_{bmt} is the measured moisture expansion strain from a 4 hour accelerated steam test and t is the age of the brick in months since 1 day after removal from the kiln. McDowall et al conducted their tests at 17 to 25 °C and 50 to 70% relative humidity. If desired, their results can be adjusted to the humidity and temperature of the 'in-situ' environment, if different than the test environment, by using the corrections given by Equations (3.52) and (3.52). Note that although McDowall implies that Equation (3.48) is only accurate for times less than 4 years, it actually only differs from Grimm's formulation (see Equation (3.50)) by about 5% for the period between 5 and 20 years.

The designer may only be interested in the amount of moisture expansion taking place after the brick has been placed in the wall or after the upper shelf angle is installed, in which case Equation (3.48) can be modified as follows;

$$\epsilon_{bm} = \epsilon_{bmt} [0.6013 \{ \ln(t_i + 2.298) - \ln(t_e + 2.298) \}] \quad (3.49)$$

where,

t_i = age of the brick, at the time of interest, in months

t_e = age of the brick, at the time of placement, in months

Grimm (40) estimates the brick moisture expansion, as a function of time, to be;

$$\epsilon_{bm} = \frac{\epsilon_{bmt}(3.12) \{ \ln(t_i + 2.3) - \ln(t_e + 2.3) \}}{3.12 \ln(t_i + 2.3) - 1} \quad (3.50)$$

where t_i is the age of brick at the time of testing in months. The value ϵ_{bmt} is derived, in this case, from a delayed moisture expansion test which involves reheating a sample of bricks to about 700 °C (in order to drive off the moisture) and measuring the amount of expansion that has occurred up until the time of the test. Hosking et al (49) and Waters et al (116) claim, though, that not all moisture expansion is removed for reheating below the original firing temperature and that additional firing shrinkages may occur. This test may, therefore, only give an approximate idea of the amount of moisture expansion which has occurred in the sample since creation.

Another disadvantage to the delayed expansion test method is that it is generally performed well after the wall has been built and so is not particularly useful as a design aid but rather as

a check on the contribution of moisture expansion to distress already present in masonry. On the other hand, if the test value can be considered reliable, it automatically takes into account the variation from such sources as firing temperatures and the temperature and humidity of the environment. If the brick unit was removed from the wall itself, the possible change in expansion due to restraint or applied stress is also taken into account.

The designer can use Grimm's relationship as a design aid by using an 'expected' strain value (and its corresponding age), derived from a source such as Ritchie (90), in place of the tested value. It should also be noted that Grimm's relationship does not agree particularly well with Ritchie's observations. However, if the base 10 logarithm is used, instead of the natural logarithm, a much closer agreement is obtained, i.e.;

$$\epsilon_{bm} = \frac{\epsilon_{bmt} \cdot (3.12) \{ \log_{10}(t_i + 2.3) - \log_{10}(t_e + 2.3) \}}{3.12 \log_{10}(t_i + 2.3) - 1.0} \quad (3.51)$$

This form of the relationship is recommended as it also yields the correct result for the case of $t_i = t_t$ (time of interest equal to the time of testing) and $t_e = 0$ (i.e., moisture expansion since removal from the kiln).

Correction Factor for the Temperature of the Environment

According to Beard et al (8), the orientation of the wall may increase the moisture expansion of the brick by as much as 30%. Thus the following correction factor is proposed for use in this study;

$$C_1 = 1 + 0.3 \cdot a \cdot \sin \left(\frac{A_s - 45}{1.5} \right) \quad (3.52)$$

where,

A_s = direction of the exterior normal to the wall measured in degrees clockwise from North

$$\begin{aligned} a &= 1.0 \quad \text{if} \quad \sin \left(\frac{A_s}{2} \right) \geq 0.3827 \quad (\text{ie. } 45^\circ \leq A_s \leq 315^\circ) \\ &= 0.0 \quad \text{if} \quad \sin \left(\frac{A_s}{2} \right) < 0.3827 \end{aligned}$$

Correction Factor for the Humidity of the Environment

Based on Ritchie's (90) observations, a correction factor for moisture expansions taking place at humidities other than 50% can be postulated to be;

$$C_2 = 0.5763 \left(\frac{R.H.\%}{100} \right)^2 + 1.7338 \left(\frac{R.H.\%}{100} \right) \quad (3.53)$$

where $RH\%$ is the relative humidity in percent

Adjustment for the Presence of Mortar Joints

Up until this point, the moisture expansion of the brick unit alone has been considered. In order to determine the total wall expansion, the effects of shrinkage in the mortar joints must also be included. Grimm (40) estimates the contraction strain in the wall due solely to the shrinkage of the mortar to be,

$$\epsilon_{ms} = \epsilon_{msu} (1 - R) (1 - 2.35 \times 10^{-7} (RH\%)^{3.3}) (0.95 + 0.008a_m)(1 + 0.05c_m) \times \left\{ \left(\frac{t_i}{t_i + 26e^{0.0142D_m}} \right) - \left(\frac{t_e}{t_e + 26e^{0.0142D_m}} \right) \right\} \quad (3.54)$$

where,

- ϵ_{ms} = shrinkage strain in the wall due solely to mortar shrinkage
- ϵ_{msu} = ultimate shrinkage strain of the mortar (see Table 3.8)
- R = ratio of modular to nominal vertical brick dimensions
- $RH\%$ = mean relative humidity of the environment in %
if $RH\% < 40\%$ use $RH_o\% = 40\%$
- a_m = air content of mortar (%)
- c_m = ratio of calcium chloride to cement content in mortar by weight (%)
- t_i = age of the mortar at the time of interest (days)
- t_e = age of the mortar when upper shelf angle was placed (days)
- D_m = maximum mortar moisture travel distance (mm), i.e., for a single wythe wall exposed on both sides, D_m equals half the wythe thickness

If air and calcium chloride contents of the mortar are not known, Grimm's relationship can be simplified to;

$$\epsilon_{ms} = \epsilon_{msu} (1 - R) (1 - 2.35 \times 10^{-7} (RH\%)^{3.3}) \left\{ \left(\frac{t_i}{t_i + 26e^{0.0142D_m}} \right) - \left(\frac{t_e}{t_e + 26e^{0.0142D_m}} \right) \right\} \quad (3.55)$$

Sahlin (93) expects mortar shrinkage to vary between 0.01% and 0.08% whereas Schubert's results (97) range from 0.04 to 0.11% with a mean value of 0.075%.

Table 3.8 Typical values of ultimate shrinkage of mortar (from Sturgeon, 1966).

Ratio of Cement : Lime : Sand	Ultimate Unrestrained Shrinkage (%)
1 : 0 : 3	0.037
1 : 0 : 6	0.028
1 : 0 : 8	0.031
1 : 1 : 6	0.035
1 : 2 : 9	0.027
1 : 3 : 12	0.017

Summary of Clay Brick Masonry Moisture Movement

The moisture expansion of the brick unit can be determined in two ways;

- 1) From McDowall and Birtwistle;

$$\epsilon_{bm} = \epsilon_{bmt} \{0.6013 \ln(t_i + 2.298) - 0.1929\} \quad (3.48)$$

$$= \epsilon_{bmt} [0.6013 \{\ln(t_i + 2.298) - \ln(t_e + 2.298)\}] \quad (3.49)$$

where,

ϵ_{bmt} = measured moisture expansion strain obtained from a 4 hour accelerated steam test

t_i = age of the brick at the time of interest, in months, since 24 hours after removal from the kiln

t_e = age of the brick at the time of placement in the wall or at the time of placement of the upper shelf angle, in months, since 24 hours after removal from the kiln

- 2) From Grimm (modified);

$$\epsilon_{bm} = \frac{\epsilon_{bmt} \cdot (3.12) \{\log_{10}(t_i + 2.3) - \log_{10}(t_e + 2.3)\}}{3.12 \log_{10}(t_i + 2.3) - 1.0} \quad (3.51)$$

where,

ϵ_{bmt} = estimated or tested brick moisture expansion strain

t_i = age of the brick at the time of testing or at the time that the estimated strain occurs, in months

If desired, the correction factor for the temperature of the environment can be calculated as follows;

$$C_1 = 1 + 0.3 \cdot a \cdot \sin \left(\frac{A_s - 45}{1.5} \right) \quad (3.52)$$

where,

A_s = azimuth of the exterior normal to the wall measured in degrees clockwise from North

$a = 1.0$ if $45^\circ \leq A_s \leq 315^\circ$
 $= 0.0$ otherwise

The correction factor for the humidity of the environment is,

$$C_2 = 0.5763 \left(\frac{R.H.\%}{100} \right)^2 + 1.7338 \left(\frac{R.H.\%}{100} \right) \quad (3.53)$$

The contribution of the mortar to the moisture strain of the wall can be found from Grimm's relationship;

$$\epsilon_{ms} = \epsilon_{msu} \cdot (1 - R) \left(1 - 2.35 \times 10^{-7} (R.H.\%)^{3.3} \right) (0.95 + 0.008a_m)(1 + 0.05\epsilon_m) \\ \times \left\{ \left(\frac{t_i}{t_i + 26e^{0.0142D_m}} \right) - \left(\frac{t_e}{t_e + 26e^{0.0142D_m}} \right) \right\} \quad (3.54)$$

$$\simeq \epsilon_{msu} \cdot (1 - R) \left(1 - 2.35 \times 10^{-7} (R.H.\%)^{3.3} \right) \left\{ \left(\frac{t_i}{t_i + 26e^{0.0142D_m}} \right) - \left(\frac{t_e}{t_e + 26e^{0.0142D_m}} \right) \right\}$$

Finally, the moisture strain in the masonry, since placement, can be found as follows;

$$\epsilon_{msR} = (R)(C_1)(C_2) \cdot \epsilon_{bm} + \epsilon_{ms} \quad (3.56)$$

at the time of interest, t_i . Care should be taken with the sign on ϵ_{ms} , letting shrinkage be negative.

3.3.4 Thermal Strain of Clay Brick Masonry

Most of the research into the coefficient of thermal expansion of clay brick seems to have been done for expansion in the horizontal direction. Sheehan (99) believes the coefficient to be isotropic, whereas Grimm (39) estimates a 22% increase for expansion in the vertical direction. Thus, for the vertical direction, Grimm gives the following relationship for the net coefficient of thermal expansion of brick masonry (see Tables 3.9 and 3.10);

$$\alpha_{br} = 1.22R \cdot C_b + (1 - R) \cdot C_m \quad (3.57)$$

where,

α_{br} = net coefficient of thermal expansion of brick masonry (vertically)

R = ratio of modular to nominal vertical brick dimensions

C_b = coefficient of thermal expansion of the brick unit

C_m = coefficient of thermal expansion of the mortar

The thermal strain in the brick masonry will be;

$$\epsilon_{th,br} = \alpha_{br} \cdot (T_{br,i} - T_{br,e}) \quad (3.58)$$

where,

$T_{br,i}$ = mean temperature of the masonry at the time of interest ($^{\circ}C$)

$T_{br,e}$ = mean temperature of the masonry at the time of construction or at the time of placement of the upper shelf angle

Table 3.9 Coefficients of thermal expansion of clay brick masonry given by various sources.

Source	Date	Brick Masonry ($\mu\text{m}/\text{m}^\circ\text{C}$)	Brick Unit		Mortar ($\mu\text{m}/\text{m}^\circ\text{C}$)	Direction
			Clay ($\mu\text{m}/\text{m}^\circ\text{C}$)	Shale ($\mu\text{m}/\text{m}^\circ\text{C}$)		
NCMA	1973	6.5				?
Bergquist	1979	6.0				Vert
BRE	1979	5.8			10.13	?
Anderson	1976	7.0				?
Schubert	1980	5.9				?
BIA	1963	6.5				?
Plummer	1962	6.5				Hor
Sturgeon	1978				6.3, 9.2	?
Loubser	1972				12.7	?
Baker	1964		5.4	5.4		?
Ross	1941		4.1, 12.4	4.7, 6.8		?
Palmer	1931		4.7, 8.5	4.1, 5.2	8.5	?
Sheehan	1976		4.1, 8.1			Both
Monk	1978		5.4	5.4		?

? - direction is not given

Table 3.10 Coefficients of thermal expansion in $\mu\text{m}/\text{m}^\circ\text{C}$ (from Grimm, 39).

Material	Low	Mean	High	Design
Brick				
Surface clay	4.1	5.9	12.4	7.70
Shale	4.7	6.1	6.8	6.59
Clay	3.1	4.0	4.7	4.30
Random	3.8	5.2	6.7	5.83
Mortar				
Cement:Lime:Sand = 1 : 1 : 6	7.51	8.05	8.60	8.28
Masonry Cement:Sand = 1 : 3	6.80	8.89	11.00	10.40

3.3.5 Freezing Expansion of Clay Brick

When clay brick freezes, it undergoes a certain amount of expansion. The variability in the magnitude of the expansion is considerable, with cracking occurring in some cases. Grimm (40) gives the mean freezing expansion of stiff mud bricks as 0.07 ± 0.03 mm/m. Davison (26) takes a much closer look at freezing expansion, as is shown in Table 3.11. Davison does not, however, indicate the direction in which the measured strains occur and therefore, in lieu of further data, freezing expansion will be assumed isotropic. The brick specimens were saturated prior to freezing and since the brick masonry on site will generally be quite dry during the winter months, the freezing strains measured by Davison will likely be larger than those encountered on site

Table 3.11 Freezing expansion strains, in mm/m, observed by Davison (26) for various brick types.

Type	Average Freezing Temperature ($^{\circ}\text{C}$)	Temporary Extreme Freezing Expansion			Permanent Freezing Expansion		
		Max.	Min.	Ave.	Max.	Min.	Ave.
Extruded Clay Brick	-7.5	1.19	0.03	0.44	0.56	0.00	0.16
Dry-Press Clay Brick	-8.1	1.03	0.32	0.63	0.59	0.07	0.31
Concrete Brick	-5.3	0.29	0.02	0.10	0.19	0.00	0.08
Sand-Lime Brick	-6.0	1.29	0.62	0.87	0.58	0.23	0.37
Sandstone	-3.2	0.53	0.30	0.45	0.44	0.21	0.37
Limestone	-3.9	0.57	0.14	0.35	0.38	0.07	0.22

3.3.6 Elasticity of Clay Brick Masonry

Being a composite material, the elastic properties of brick masonry are dependent on, not only the brick properties, but also the mortar properties. To determine the modulus of elasticity of brick masonry by considering the elastic properties of the brick and mortar separately and then combining them is complex and possibly unjustified (considering the variability of each material). This becomes particularly true if the change of elasticity with time is also included in the model.

Many empirical relationships have been suggested by various researchers over the years in which the elastic modulus is related to the compressive strength of either the brick unit or the masonry as a whole. Probably the most concise relationship found was that proposed by Grinn (40) in which the elastic modulus can be predicted in two ways,

1) If test results are known,

$$E_{MBR}(t_i) = E_{test} \left[\frac{\ln(f'_{m_{test}}) + \ln \left(\frac{31 + \ln(t_i)}{31 + \ln(t_{test})} \right) - 1.12}{\ln(f'_{m_{test}}) - 1.12} \right] \quad (3.59)$$

where,

E_{test} = tested modulus of elasticity (MPa)

$f'_{m_{test}}$ = tested compressive strength of masonry (MPa)

t_{test} = age of the brick masonry at the time of testing (days)

t_i = age of the brick masonry at the time of interest (days)

2) If material properties are known,

$$\begin{aligned} f'_m(t_i) &= (6.895 \times 10^{-3})(3.37 + \ln(c/s))(\ln(f'_{br}) - 2.253) \left(273 - \left(\frac{R}{1-R} - 14 \right)^2 \right) \\ &\quad \times (31 + \ln(t_i))(0.05c_m)(57.3 - a_m)^{0.33} \quad (3.60) \\ &= 35 \text{ MPa} \quad \text{for Sarabond Mortar} \end{aligned}$$

$$\begin{aligned} E_{MBR}(t_i) &= 7.9565 \times 10^3 [\ln\{f'_m(t_i)\} - 1.12] \quad (3.61) \\ &= E_{MBR}(28) + 7.9565 \times 10^3 [\ln\{31 + \ln(t_i)\} - 3.536] \end{aligned}$$

where,

c/s = volumetric ratio of cement to sand in mortar

= 0.33 for type M mortar

= 0.22 for type S mortar

= 0.17 for type N mortar

= 0.11 for type O mortar

R = ratio of modular to nominal vertical brick dimensions

a_m = air content of mortar in percent
 = 20% if air entrainment or masonry cement used
 = 4% otherwise

f'_{br} = compressive strength of brick unit (MPa)
 if f'_{br} is greater than 96.5 MPa, use $f'_{br} = 96.5$ MPa in Eq. (3.60)

c_m = workmanship coefficient
 = 1.0 if observed

$$= \frac{140.8}{261.2 - f'_{br}} \quad \text{if unobserved} \quad (3.62)$$

if f'_{br} is greater than 123 MPa, use $f'_{br} = 123$ MPa in Eq. (3.62)

Note that the value of f'_m begins to decrease for values of R exceeding 0.93. This does not really make sense, and so R should be limited to a maximum value of 0.93.

Although there is a significant variation in f'_m with time, the elastic modulus varies with the double logarithm of time, and so grows very slowly - about 6% in 5 years. For the same interval, Poljakov (86) predicts a variation in $EMBR$ of about 66% while concrete elasticity varies by about 27%. Because of the nature of brick masonry, the variation in its elastic modulus would be expected to be less than that observed in concrete (due primarily to hardening of the mortar) and so Grimm's estimation appears to be in the correct ballpark. However, since the variation is so slight, it can be omitted in most cases.

Eskanasi (33) presents relationships for both the elastic modulus and the compressive strength of brick masonry, which, although they do not include any variation with time, are of particular relevance here because they were developed in Canada.

$$EMBR(28) = 220.6 f'_{br} + 1000 \quad \text{MPa} \quad (3.63)$$

$$f'_m(28) = 3.361 \sqrt{f'_{br}} - 1.002 \sqrt{d} + 15.31 \quad \text{MPa} \quad (3.64)$$

where d is the wall thickness in mm. Equations (3.63) and (3.64) are valid for values of d less than 200 mm and f'_{br} greater than 40 MPa.

3.3.7 Corrosion Expansion of Steel Shelf Angles

The formation of corrosion products between the masonry and the supporting shelf angle will force the masonry to move upwards (or, conversely, the shelf angle will be forced down). If the shelf angle is adequately protected by coatings, flashing, and weepholes, then the corrosion will be negligible. However, if the steel is intermittently exposed to water then corrosion will occur.

Grimm (40) estimates the effective strain in the masonry due to corrosion expansion to be,

$$\epsilon_{cs} = (K_c - 1.0) \left(\frac{\Delta d_c}{h} \right) \left[\frac{t_i - t_{sp}}{t_{sa} - t_{sp}} \right]^{0.3} \quad (3.65)$$

where,

ϵ_{cs} = effective strain in the veneer due to corrosion of the steel shelf angle

K_c = ratio of the volume of iron oxide to the original volume of iron (ranges from 1.4 to 4.1 with a mean value of 2.1)

Δd_c = shelf angle thickness reduction due to corrosion occurring between the times t_{sp} and t_{sa} in mm

h = height of the supported masonry panel in mm

t_i = time of interest (days)

t_{sp} = time at which the shelf angle was placed (days)

t_{sa} = time at which the thickness reduction was measured (days)

Equation (3.65) depends on the measurement of the amount of corrosion which has occurred some time after the placement of the shelf angle. In many cases this is not feasible and the designer must estimate Δd_c based on past experience. Stetina (104) estimates the average rate of unprotected steel surface corrosion to be about 0.13 mm per year when exposed to generous amounts of water and oxygen. This rate may drop to about 0.01 mm per year if the surface is left relatively dry after the first year.

3.4 Concrete Block Masonry

3.4.1 Creep of Concrete Block Masonry

Very little research has been done, particularly in Canada, on the creep of concrete blockwork. Of the work that has been done, though, of particular note is Lenczner's research (85) about which the following observations can be made;

- 1) Test piers made of low strength concrete blocks (5.8 MPa) were loaded at 28 days to stress levels ranging from 18 to 45% of the blockwork strength.
- 2) Approximately 20% of the creep occurred in the mortar joints, implying that creep strains in the mortar were 4 to 5 times that occurring in the block itself.
- 3) For all stress levels, creep effectively ceased after 300 days.
- 4) The ratio of maximum load strain to instantaneous strain decreased from 7.6 at the lowest stress level (stress/strength = 0.18) to 4.3 at the highest stress level (stress/strength = 0.45). These values correspond to creep coefficients of 6.6 and 3.3 respectively.
- 5) The use of such low strength block by Lenczner renders the results of his research questionable in Canada since Canadian practice involves the use of much higher strength block (13 MPa and greater).

Research conducted by NCMA (76) illustrates the difference between Lenczner's observations and those made in North America on high strength block (30 to 45 MPa). It appears, from Figure 3.12, that the strain ratios are not only dependent on the stress/strength ratio but also on the block composition or strength. Note that the block strengths used in the NCMA tests are somewhat higher than those used in normal Canadian practice.

Jessop (55) states that the creep in concrete masonry units is of approximately the same order of magnitude as creep in ordinary concrete made from similar aggregates. Also since creep in the mortar joints exceeds that of the block unit, by a factor of 4 to 5 as reported by Lenczner, creep in the wall will exceed creep in the block units by 20 to 40%.

Tatsa (111) found that the specific creep in concrete blocks varied from 7.7 to 9.3 $\mu\text{m}/\text{m}/\text{MPa}$, depending on environmental conditions, at an applied load of 45% of the block masonry compressive strength. The ratio of creep in the joints to creep in the block was found to vary from 4.4 to 16.8. NCMA (76) found the specific creep of the concrete masonry as a whole (using high strength block) to be about 48 $\mu\text{m}/\text{m}/\text{MPa}$ whereas Buchanan (16) observed specific creeps in the masonry between 88 and 91 $\mu\text{m}/\text{m}/\text{MPa}$ for ungrouted masonry and between 95 and 123 $\mu\text{m}/\text{m}/\text{MPa}$ for grouted masonry.

2

Figure 3.12 Creep strain observations made by Lenczner on low strength block versus NCMA observations on high strength block (from Sturgeon, 1966).

Estimation of the Magnitude of Creep

The only mathematical relationship found between creep, applied stress, and time for concrete blockwork was proposed by Lenczner (65). Lenczner uses a modified Kelvin model to relate the visco-elastic behaviour of a concrete block wall with time as follows;

$$\epsilon_{crBL} = \epsilon_i \left(2.5 - 1.2e^{-\frac{Et}{\lambda}} \right) \quad (3.66)$$

where,

ϵ_{crBL} = creep strain at the time t , in days, after application of the load

ϵ_i = instantaneous elastic strain

E = elastic modulus of the wall (MPa)

λ = linear coefficient of viscosity (MPa-days)

Lenczner derives the value of λ to be 1.435×10^8 MPa-days. Substitution of results from the NCMA tests for blockwork loaded at 28 days into Equation (3.66) yields a λ value of 1.45×10^7 MPa-days

which is ten times larger than Lenczner's result. Other drawbacks to Lenczner's relationship can be stated as follows;

- 1) no age at loading adjustment factor is provided,
- 2) experimental data is required to evaluate λ ,
- 3) the constants 1.2 and 2.5 may not be applicable to Canadian materials,
- 4) the relationship seems to be overly sensitive to changes in the elastic modulus, E , which is a highly variable property.

Let us now consider the creep relationships which have already been derived for concrete in Section 3.2.1, and check to see if they satisfy some of the observations that have been made about blockwork behaviour - particularly with respect to tests carried out at Carleton University by Buchanan (16). Thus, for a stepwise varying applied stress we have;

$$\epsilon_{crBL}(t_i) = \sum_{j=1}^{(i-1)} k_j \cdot (\sigma_{M,j} - \sigma_{M,j-1}) \cdot \Phi_M(t_i, t_j) \quad (3.67)$$

where,

ϵ_{crBL} = total unrestrained creep strain in the concrete blockwork at the time of interest, t_i

$\sigma_{M,j}$ = total stress, in MPa (based on net area), applied to the blockwork concrete at the beginning of interval j , compression being negative

k_j = creep recovery correction factor

= 1.0 if $(\sigma_{M,j} - \sigma_{M,j-1}) \leq 0$

= 0.67 if $(\sigma_{M,j} - \sigma_{M,j-1}) > 0$

t_i = age of the concrete blockwork at the time of interest (days)

t_j = age of the concrete blockwork at the beginning of interval j (days)

$\Phi_M(t_i, t_j)$ = blockwork creep function giving the specific creep, in mm/mm/MPa, at the time of interest, t_i , due to the stress increment applied at the beginning of interval j

Note that for an applied stress which is constant (as is the case in both the Buchanan and NCMA tests), Equation (3.67) simplifies to;

$$\epsilon_{crBL}(t_i) = \sigma_j \cdot \Phi_M(t_i, t_i) \quad (3.68)$$

in which t_j now becomes t_i , which is the age of the masonry at the time of loading in days. The blockwork creep function is similar to that given by Equation (3.2) and appears as follows;

$$\Phi_M(t_i, t_j) = \frac{1}{EMBL(28)} [\beta_a(t_j) + \psi_d \cdot \beta_d(t_i - t_j) + \psi_f \{ \beta_f(t_i) - \beta_f(t_j) \}] \quad (3.69)$$

where $EMBL(28)$ is the 28 day modulus of elasticity of the blockwork in MPa and the other terms are as defined in Section 3.2.1. In the calculation of $\beta_a(t_j)$, it is assumed that the compressive strength change in block masonry is identical to that of concrete. A comparison with NCMA observations (76), made in Section 3.4.4, shows reasonably good agreement with the sparse data available.

Substitution of Buchanan's elastic, geometric, and environmental parameters for grouted blockwork into the creep function given by Equation (3.69), yields the predicted creep strain values (from Equation (3.68)) shown in Table 3.12. It can be seen that the predicted creep strains lie well within the error commonly associated with a creep analysis of this kind ($\pm 20\%$ at its best).

If the same analysis is carried out with the ungrouted blockwork tests made by Buchanan, the predicted creep strains can be compared to the observed, as shown in Table 3.13.

Table 3.12 Comparison of predicted creep strains at 70 days with those observed by Buchanan (16) for grouted blockwork.

Test Series	G1	G2
Applied Stress	3.67 MPa	3.49 MPa
Observed Creep	350 $\mu m/m$	430 $\mu m/m$
Predicted Creep	380 $\mu m/m$	375 $\mu m/m$
Difference	8.5%	-12.8%

Table 3.13 Comparison of predicted creep strains at 70 days with those observed by Buchanan (16) for ungrouted blockwork.

Test Series	UG1	UG2
Applied Stress	4.94 MPa	3.86 MPa
Observed Creep	450 $\mu\text{m}/\text{m}$	340 $\mu\text{m}/\text{m}$
Predicted Creep	571 $\mu\text{m}/\text{m}$	390 $\mu\text{m}/\text{m}$
Difference	+27%	+15%

The creep strains derived from the C.E.B. relationships appear, from Table 3.13, to be somewhat high. Although it is difficult to suggest improvements on the theory used on the basis of just two short term tests, some thoughts are as follows;

- 1) If the strict definition for d_m is applied to blockwork, ie;

$$d_m \propto \frac{(\text{concrete area})}{(\text{perimeter})}$$

then the introduction of a hollow core will result in the concrete area decreasing while the perimeter (air-concrete interface) increases. The net result is a decrease in d_m which in turn causes an increase in the expected creep. Since this does not agree with the observed creep strains and there does not appear to be any rational reason for increasing d_m , d_m will be left as it is presently defined (Equation (3.8)).

- 2) A more likely explanation for the lower than expected creep strains is that the age of the block unit is in fact quite a bit more than the age of the masonry. As the age of a concrete sample increases, the creep that it will experience under a given load will decrease. To assume that t_i is equal to, say, 14 days implies that both the mortar and the block were mixed and cast 14 days prior to loading. Since the block is often considerably older, it will experience less creep than will the mortar (agreeing with Lenczner's observations).

Taking the latter into account in the creep analysis may be somewhat difficult as its effect on creep depends not only on the relative mortar joint thickness to block height, but also on the percentage of cores grouted and the 'percent solid' of the block itself.

In summary, it appears that the creep of blockwork should be calculated using the C.E.B. relationships as outlined in Section 3.2.1 (Creep of Concrete) with the following provisions;

- 1) If the wall is ungrouted, use the true age of the block prior to laying and add it to t_i and t_j .
- 2) For fully grouted walls, assume that the block age is the same as that of the block masonry (no adjustment on t_i and t_j).

- 3) For partially grouted walls, calculate an effective age of the block unit prior to laying as follows;

$$\ln(t_{eff} + 1) = \left[\frac{0.44 - \alpha(1 - \psi)}{0.44} \right] \cdot \ln(t_M + 1) \quad (3.70)$$

where,

t_{eff} = effective age of the block unit at the time of construction (days)

t_M = true age of the block unit at the time of construction (days)

α = fraction of cores grouted

ψ = ratio of the net block area to the gross block area

Equation (3.70) is based on the approximate relationship between the growth of creep strain and the logarithm of time and is really only strictly applicable to the standard modular block ($\psi = 0.56$). The values of t_i and t_j can now be adjusted by adding t_{eff} to each.

- 4) For a wall whose horizontal length is much greater than its thickness, the calculation of d_m can be simplified to;

$$d_m = \lambda \cdot d_w \quad mm \quad (3.71)$$

where d_w is the thickness of the wall in mm.

A further comparison of creep strains, as determined by these guidelines, with tests performed by NCMA (76) on blockwork can be found in Appendix D.

3.4.2 Shrinkage of Concrete Block Masonry

The ultimate shrinkage strain arising in concrete block units is dependent on the type and length of curing, the aggregate type, and the relative humidity of the environment. NCMA (73) gives values of shrinkage of concrete block, for various aggregate types, ranging from 0.13 to 0.90 mm/m based on the ASTM C-426-58T test for drying shrinkage of concrete block. An average value of 0.38 mm/m is given corresponding to a mean relative humidity of 22% (note that this is an accelerated test).

The variation of shrinkage with relative humidity seems to be generally linear with one point defined by the ultimate observed shrinkage value at its corresponding relative humidity, and the other by zero shrinkage and 100% relative humidity. It is uncertain how representative the ASTM 426 test is of the actual ultimate shrinkage, particularly with respect to varying relative humidity. Table 3.14, though, indicates very good agreement between the ASTM 426 test and normal shrinkage at 70% relative humidity.

Table 3.14 Comparison of accelerated ASTM C-426 shrinkage test values with slow drying shrinkage values of concrete block (from Sturgeon, 106).

Shrinkage Group		ASTM C-426		Slow Drying	
Aggregate	Cure	Average Shrinkage, %	No. of Tests	Average Shrinkage, %	No. of Tests
Normal Weight	HP	0.019	36	0.017	16
	LP	0.027	37	0.025	15
Light Weight	HP	0.023	96	0.024	104
	LP	0.042	104	0.040	99
Pumice	HP	0.039	5	0.039	4
	LP	0.063	24	0.059	7
		0.033	302	0.031	245

HP=high pressure steam cured

LP=low pressure moist cured

The shrinkage of concrete block walls is more complicated than just simply applying the shrinkage strain of the block unit to the wall. BIA (13) reports 25 to 30% more shrinkage in walls than in the individual units, whereas Hedstrom (45) observed somewhat less shrinkage in walls than in the blocks. Hedstrom attributes his findings to restraint offered by the mortar, but no rational explanation is offered by BIA for its findings.

Table 2.15 Summary of unrestrained shrinkage strains observed in concrete block wallettes by BIA (18) in percent.

Aggregate	Steam Cure	Condition When Laid	
		Saturated	Conditioned at 70% R.H.
Sand-gravel	HT	0.033	0.023
	HP	0.023	0.009
Expanded shale	HT	0.057	0.031
	HP	0.034	-
Cinder	HT	0.069	0.045
	HP	0.047	0.023
Expanded slag	HT	0.062	0.034
	HP	0.030	-

HT = high temperature

HP = high pressure

The time lapse between block production and being laid in the wall has an unclear effect on wall shrinkage. Allan (1), Bryson (15), and Valore (114) all report reduced shrinkage with increased storage period. When the block cores are grouted, though, a considerable amount of new concrete is introduced in the vertical direction. In this situation, the grouted cores will shrink more than the older 'preshrunk' shells, which tends to place the shells in compression and the cores in tension. Ideally, this should be handled by calculating the block and core shrinkages separately, and then using compatibility and elastic relationships to find the 'net' shrinkage. Furthermore, if the elasticities of the shell and grout can be considered approximately equal, the net shrinkage becomes simply a weighted average of the shell and grout shrinkages (the 'weighting' being dependent on the relative cross-sectional areas of the shell and grout).

It should also be noted that the addition of grout adds a considerable amount of moisture to the block, causing some reversible expansion, and subsequent shrinkage, in the shell (in addition to the normal shrinkage which would occur). HHFA (51) found that rewetting the concrete block resulted in an expansion of 1/6 to 1/3 of the shrinkage which had already occurred in the block (assume an average of 1/4).

Prediction of Block Masonry Shrinkage

As stated previously, the shrinkage strain in the vertical direction of concrete block masonry will be assumed to be a weighted function of the individual grout and shell shrinkages;

$$\epsilon_{shBL}(t_i) = \left[\frac{\alpha(1-\psi)}{\alpha(1-\psi)+\psi} \right] \cdot \epsilon_{shGROUT}(t_i) + \left[\frac{\psi}{\alpha(1-\psi)+\psi} \right] (R_M) \cdot \epsilon_{shBLOCK}(t_i, t_o) + \left[\frac{\psi}{\alpha(1-\psi)+\psi} \right] (1-R_M) \cdot \epsilon_{shMORT}(t_i) \quad (3.72)$$

where,

- $\epsilon_{shBL}(t_i)$ = vertical block masonry shrinkage strain at the time t_i
- t_i = age of the masonry at the time of interest (days)
- t_o = age of the block unit at the time of construction (days)
- R_M = ratio of the modular to nominal vertical block dimensions
- α = fraction of cores grouted
- ψ = ratio of the net to gross block unit area

The grout shrinkage strain, $\epsilon_{shGROUT}(t_i)$, can be found using the concrete shrinkage strain relationship as follows;

$$\epsilon_{shGROUT}(t_i) = - \left[7.7206 \times 10^{-4} \left(1 - \frac{R.H.\%}{100} \right)^{0.7787} \right] \left[0.6 + \left(\frac{18}{d_m} \right)^{0.6} \right] \cdot \beta_S(t_i) \quad (3.73)$$

where,

- $\epsilon_{shGROUT}(t_i)$ = unrestrained shrinkage strain of the grouted core
- $R.H.\%$ = mean relative humidity of the environment (%)
- d_m = effective block wall dimension
= $\lambda \cdot d_o$ as defined in Section 3.4.1
- $\beta_S(t_i)$ = variation of shrinkage with time correction factor, as defined in Section 3.3.2

Alternatively, if the ultimate shrinkage strain of the grout is known, Equation (3.73) can be simplified to;

$$\epsilon_{sh\text{GROUT}}(t_i) = \epsilon_{sh\text{GROUT}} \cdot \beta_g(t_i) \quad (3.74)$$

where $\epsilon_{sh\text{GROUT}}$ is the known ultimate shrinkage strain of the grout.

For a relative humidity of 70%, the ultimate shrinkage strain predicted by Equation (3.73) is 0.03% which agrees very well with the average values of concrete block shrinkage given in Table 3.14. It is recommended, therefore, that Equation (3.73) be used to predict the block unit shrinkage, if unknown, with some modifications as follows;

$$\epsilon_{sh\text{BLOCK}}(t_i, t_o) = \epsilon_{sh\text{ULT}} \cdot \left[\beta_g(t_i) - (1 - 0.25\alpha) \cdot \beta_g(t_o) \right] \quad (3.75)$$

where,

$\epsilon_{sh\text{ULT}}$ = ultimate shrinkage strain of the block unit (if unknown, use Equation 3.76)

$$= - \left[7.7206 \times 10^{-4} \left(1 - \frac{R.H.\%}{100} \right)^{0.7737} \right] \left[0.6 + \left(\frac{18}{d_m} \right)^{0.5} \right] \quad (3.76)$$

The amount of moisture expansion due to wetting is taken into account through the term $(1 - 0.25\alpha)$, where the assumption is that the degree of wetting is directly proportional to the fraction of cores grouted. If the block is wetted before laying, use 0.75 in place of the term $(1 - 0.25\alpha)$.

Shrinkage strain in the mortar joints should also be considered, particularly if the cores are not grouted. As outlined in Section 3.3.3, Grimm (40) proposes the shrinkage of mortar to be;

$$\epsilon_{sh\text{MORT}}(t_i) = (\epsilon_{mou}) (1 - 2.35 \times 10^{-7} (R.H.\%)^{2.3}) (0.95 + 0.008a_m) \\ \times (1 + 0.05c_m) \left(\frac{t_i}{t_i + 20e^{0.0142D_m}} \right) \quad (3.77)$$

where,

$\epsilon_{sh\text{MORT}}(t_i)$ = unrestrained shrinkage strain of the mortar at the time of interest t_i

ϵ_{mou} = ultimate shrinkage strain of the mortar (see Table 3.8)

a_m = air content of mortar (%)

c_m = ratio of calcium chloride to cement content in the mortar by weight (%)

t_i = age of the masonry at the time of interest (days)

D_m = maximum mortar moisture travel distance (mm)

$$= \frac{1}{2} \cdot d_w \text{ generally}$$

d_w = wall thickness (mm)

If the air and calcium chloride contents are not known, Equation (3.77) can be simplified to;

$$\epsilon_{sh\,MORT}(t_i) = (\epsilon_{msu}) (1 - 2.35 \times 10^{-7} (R.H.\%)^{3.3}) \left(\frac{t_i}{t_i + 26e^{0.0142D_m}} \right) \quad (3.78)$$

3.4.3 Thermal Strain of Concrete Block Masonry

Because the materials grout, mortar, and concrete block are all very similar, it is probably justifiable to ignore the individual thermal strains and consider the block unit strain as representative of the total strain in the wall. If, however, the coefficient of thermal expansion of the block unit is considerably different than that of the grout or mortar, then the individual strains must be calculated and combined in the same manner as proposed for the shrinkage strain (see Section 3.4.2, Equation (3.72)).

For a concrete or steel frame building, where the exterior concrete masonry wall usually appears as an infill panel, the masonry thermal strains are generally quite straight forward to determine once the thermal gradient through the wall is known.

$$\epsilon_{th\,BL} = \alpha_M \cdot (T_{M,i} - T_{M,e}) \quad (3.79)$$

where,

$\epsilon_{th\,BL}$ = vertical thermal strain in the blockwork panel

α_M = coefficient of thermal expansion of the block in $m/m/^{\circ}C$ (see Tables 3.16 and 3.17)

$T_{M,i}$ = mean temperature of the blockwork at the time of interest ($^{\circ}C$)

$T_{M,e}$ = mean temperature of the blockwork at the time of its construction or at the earlier time of interest ($^{\circ}C$)

In a loadbearing masonry structure, however, there is often a return wall which is keyed into the exterior blockwork wall. Since the return wall, surrounded by the interior environment, is at a relatively constant temperature, it tends to restrain the movements of the exterior wall. The extent of this restraint, and the resulting 'net' coefficient of thermal expansion of the exterior wall is unknown at this time. Barring further study, the designer could possibly take this effect into account by estimating a 'net' coefficient of thermal expansion for the exterior wall based on such parameters as the location of the insulation and the relative cross-sectional areas of the exterior and return walls.

Table 3.16 Coefficients of thermal expansion of blockwork for various aggregate types (from Sturgeon, 196).

Aggregate Type	Coefficient of Thermal Expansion ($\mu\text{m}/\text{m}/^{\circ}\text{C}$) Over the Temperature Interval Indicated			
	23 to 62 $^{\circ}\text{C}$	23 to 100 $^{\circ}\text{C}$	-21 to 52 $^{\circ}\text{C}$	-40 to 60 $^{\circ}\text{C}$ Ottawa Region
Cinders	5.6	5.0	3.8	
Expanded Shale	7.7	7.4	7.0	
Expanded Slag	8.3	7.6	7.0	7.6
Volcanic Pumice	7.4	6.8	-	6.8
Sand and Gravel	9.4	8.6	9.9	9.5
Crushed Limestone	-	-	9.2	9.2
Haydite	-	-	7.0	
Pottisco	-	-	9.0	
Superoak	-	-	7.0	
Waylite	-	-	7.9	
Scoria	6.3	-	-	

Table 3.17 Coefficients of thermal expansion of blockwork according to the sources indicated.

Source	Date	Aggregate	Coefficient ($\mu\text{m}/\text{m}/^{\circ}\text{C}$)
Jessop (56)	1980		6 to 13
BRE (17)	1979	Normal Weight	6 to 12
		Light Weight	8 to 12
BIA (13)	1963	Dense	9.4
		Cinder	5.6
		Expanded Shale	7.7
		Expanded Slag	8.3
		Volcanic Pumice	7.4

3.4.4 Elastic Properties of Concrete Block Masonry

As with brick masonry, blockwork is a composite material whose elastic properties are dependent on the individual properties of the block unit, the mortar, and the grout (if present). According to work performed by Drysdale (28), the elastic moduli of grouted prisms were all substantially less than those of hollow prisms, even for grout strengths in excess of the block unit strengths.

Because the presence of grout makes a difference in the elastic properties of the assemblage, these properties will be considered for the grouted and ungrouted blockwork cases separately.

Modulus of Elasticity of Ungrouted Masonry

The most popular approach to the determination of the elastic modulus of blockwork is to relate the elastic modulus to the compressive strength of the masonry, f'_m . Ekanasi et al (33) developed a regression equation to predict the initial tangent modulus of ungrouted blockwork (based on the net area) as follows;

$$E_{ugBL} = 440f'_m + 5040 \text{ MPa} \quad (3.80)$$

where,

E_{ugBL} = modulus of elasticity of ungrouted block masonry in MPa

f'_m = ultimate uniaxial masonry compressive strength, based on net area, in MPa

The dispersion of data about this line was considerable, and so an improved relationship was suggested by Sturgeon (105) to be;

$$E_{ugBL} = 432f'_m + 4668 \text{ MPa} \quad (3.81)$$

which shows less variation ($r = 0.86$). Ekanasi also presents a relationship between masonry strength and the individual block and mortar strengths which shows excellent agreement with 191 tests ($r = 0.962$);

$$f'_m = 0.644f'_b + 1.184\sqrt{f'_{mort}} - 3.405 \text{ MPa} \quad (3.82)$$

where,

f'_b = ultimate block unit compressive strength, based on net area, in MPa

f'_{mort} = ultimate mortar compressive strength, in MPa

A simplified relationship was also suggested having a correlation coefficient (r) of 0.948;

$$f'_m = 0.69 f'_{bl} - 0.217 \text{ MPa} \quad (3.83)$$

Another method of determining the modulus of elasticity was suggested by Sahlin (93) involving the calculation of the individual moduli of elasticity of the block and mortar, and combining them as follows;

$$E_{ugBL} = \frac{1}{\left(\frac{1-R_{bl}}{E_{mort}}\right) + \left(\frac{R_{bl}}{E_{bl}}\right)} \quad (3.84)$$

where,

R_{bl} = ratio of modular to nominal vertical block dimensions

E_{mort} = modulus of elasticity of the mortar (MPa)

E_{bl} = modulus of elasticity of the block unit (MPa)

Sturgeon (106) found the best regression equation relating block unit strength and elasticity, based on work by Richart et al (89), to be;

$$E_{bl} = 765 f'_{bl} + 1690 \text{ MPa} \quad (r = 0.73) \quad (3.85)$$

The modulus of elasticity of mortar is suggested by Sturgeon (105) to be;

$$E_{mort} = 1000 f'_{mort} \text{ MPa} \quad (3.86)$$

which shows good agreement with experimental data based on 50 mm cubes. There is some suspicion, however, that the effect of contact with the block unit may make a substantial difference in the elastic properties of the mortar. Sturgeon also showed that the elastic moduli of grout in contact with the block averages only about 56% of the value determined from standard moist cured cylinder tests. If the same phenomenon can be assumed to occur with the mortar then we have;

$$E_{mort} = 560 f'_{mort} \text{ MPa} \quad (3.87)$$

or if the C.E.B. relationship for concrete is applied to the mortar;

$$\begin{aligned} E_{mort} &= (0.56) \left(11875 \sqrt[3]{f'_{mort}} \right) \text{ MPa} \\ &= 6650 \sqrt[3]{f'_{mort}} \end{aligned} \quad (3.88)$$

Modulus of Elasticity of Grouted Masonry

For fully or partially grouted blockwork walls, Sturgeon (105) proposes the following relationship for the calculation of the elastic moduli;

$$E_{MBL} = \frac{[(0.56)(E_{grout})(\alpha)(1 - \psi)] + [(E_{ugBL})(\psi)]}{(\alpha)(1 - \psi) + \psi} \quad (3.89)$$

where,

E_{MBL} = modulus of elasticity of fully, partially, or ungrouted block masonry, in MPa

E_{grout} = modulus of elasticity of grout as determined from standard moist cured cylinders, in MPa

$$= 11875 \sqrt[3]{f'_{grout}} \text{ MPa} \quad \text{from C.E.B., 24} \quad (3.90)$$

E_{ugBL} = ungrouted blockwork elastic moduli (MPa)

α = fraction of cores grouted

ψ = ratio of the net to gross block unit area

Using experimentally determined values for E_{grout} and E_{ugBL} , Equation (3.89) shows excellent agreement with the average experimental moduli of the grouted blockwork tested by Sturgeon, as shown in Figure 3.13.

Figure 3.13 Observed versus predicted prism elastic modulus, using Equation (3.89) (from Sturgeon, 105).

Variation of Elasticity with Time

The variation of the strength or elastic modulus of block masonry with time is not well understood. NCMA (76) provides some test data but make no attempt to develop mathematical relationships. The time relationships derived from C.E.B. (24) in Section 3.2.3 are;

$$f'_m(t_i) = f'_m(28) \cdot \left[0.31029 \ln(1.16306t_i) - 0.30730 \ln \left(1 + \frac{t_i}{93.09} \right) \right] \quad (3.91)$$

$$E_{MBL}(t_i) = E_{MBL}(28) \cdot \left[0.31029 \ln(1.16306t_i) - 0.30730 \ln \left(1 + \frac{t_i}{93.09} \right) \right]^{\frac{1}{4}} \quad (3.92)$$

where,

f'_m = the compressive strength of the block masonry at the time indicated, in MPa

E_{MBL} = the modulus of elasticity of the block masonry at the time indicated, in MPa

= E_{gBL} or E_{uBL}

t_i = age of the block masonry at the time of interest (days)

The observations made by NCMA (76) have been plotted in Figure 3.14 and the C.E.B. relationships overlaid. The observed data appears to have some inconsistencies, but if these are assumed to be due to normal variation, the trend indicates an increase in elastic moduli and strength with time. The C.E.B. relationships show reasonable agreement with the NCMA observations, particularly for the grouted prisms.

Figure 3.14 Time variation in compressive strength and elastic moduli of concrete block masonry prisms (from NCMA, 76).

3.5 Steel

The material properties of steel, be it structural or reinforcing, are well known;

$$\alpha_{st} = 12.06 \times 10^{-6} \text{ mm/mm}^\circ\text{C} \text{ (coefficient of thermal expansion)}$$

$$E_{st} = 200\,000 \text{ MPa (elastic modulus)}$$

The unrestrained thermal strain in the steel (ϵ_{st}) at the time of interest can be determined to be;

$$\epsilon_{st} = \alpha_{st} \cdot (T_{st,i} - T_{st,e}) \quad (3.93)$$

where,

$T_{st,i}$ = mean temperature of the steel at the time of interest ($^\circ\text{C}$)

$T_{st,e}$ = mean temperature of the steel when placed, or at the earlier time of interest, in $^\circ\text{C}$

Chapter 4

Veneer - Frame Interaction

In the previous chapter, the magnitudes of the unrestrained movements of each of the building elements were predicted. If these movements are not properly accounted for in the design of the building, they may result in distress of elements not intended to carry load. In particular, the effect of differential movements on the stress incurred in the veneer of a building will be investigated in this chapter.

To enable the evaluation of creep stresses, ie. stresses which are relatively constant throughout each time interval considered, the entire analysis will be done under average conditions. The conditions which tend to fluctuate in a building, causing changes in the stress levels of the structural elements, are temperature and live loading. These will be set to values which can be considered to be average throughout the time interval considered.

At certain specified times the analysis can be repeated to calculate the stresses caused by 'extreme' conditions, ie. extreme temperatures and higher live load levels (as specified by the user). These conditions are assumed to act only at the specified instant in time, and so the stresses derived from them are not applicable to the calculation of creep strains.

In this chapter, the veneer-frame interaction analysis will be discussed for the 'average' condition only, with shelf angles (or 'lumped' ties in loadbearing masonry structures) at every level. The reader should note, however, that the computer program, developed out of the theory presented here, will also handle the 'extreme' conditions discussed above as well as multiple level shelf angle spacing.

4.1 Shelf Angle Shear Analysis

When differential movements take place between the frame of a building and the veneer, they may result in deformations of the connectors linking the two. For most steel and reinforced concrete structures, the connectors (or supports) are steel shelf angles placed at intervals along the height of the building (usually at each level). Loadbearing masonry structures, on the other hand, usually employ metal ties at, for example, every second block course over the height of the building. For the purpose of this analysis, these ties will be considered to act as shelf angles, as if 'lumped' at the slab level of each storey.

The deformation of the shelf angle results in a 'shear' force set up in the angle which manifests itself as an axial force acting on the veneer in one direction, and on the frame in the other. The magnitude of the shear force depends on;

- 1) the magnitude of the relative deformations between the frame and the veneer,
- 2) the size of the expansion gap left below the shelf angle,
- 3) the stiffness and yield point of the shelf angle,
- 4) the vertical stiffness of the veneer and the frame (both are assumed to be fully elastic over the range of stresses expected),
- 5) the relative movements occurring in the levels above and below the level being considered.

The last factor can be best explained by a simple example, as shown in Figure 4.1, in which the relative deformations at level l result in the brick panel at level $(l + 1)$ being forced upwards into contact with its upper shelf angle. If this interaction had not been considered, the compressive stress in the brick panel at level $(l + 1)$ may have been predicted to be zero.

Figure 4.1 Veneer-Frame interaction model.

In order to consider the interaction taking place between all levels, either a system of simultaneous equations or an iterative analysis must be used. Although an iterative analysis is unwieldy and slow, it has the advantage of being able to include non-linearities in the system, i.e. the proposed model should consider;

- 1) yielding of the shelf angles,
- 2) changing stiffness of the frame due to the slab lifting off the infill blockwork (this will occur if the cumulative shear forces in the shelf angles are sufficient),
- 3) the inability of the shelf angles to exert an upwards force on the underlying brick panel (i.e. if the brick is not in contact with the upper shelf angle, no stress is incurred in the panel).

The iterative shelf angle shear analysis will consist of the following steps within each time interval;

- 1) Calculate the net frame deformation at each level.
- 2) Calculate the net unrestrained brick veneer deformation at each level.
- 3) Calculate the flexibility of the frame and veneer at each level at this time.
- 4) Calculate the 'at rest' position of the upper shelf angle at each level (including the deformation due to the shear forces transmitted from above according to the previous iteration).
- 5) Calculate the unrestrained position of the upper surface of the brick veneer panel at each level.
- 6) If the positions calculated in steps 4) and 5) overlap, then calculate the deformed positions of both the upper and lower shelf angles and the resulting shear forces at each level.
- 7) Check to see if either (or both) the upper or lower shelf angle yields. If so, introduce the appropriate limiting force(s) and recalculate the deformed positions of both shelf angles, assuming plastic behaviour.
- 8) Check to see if the frame flexibility changes due to liftoff from the blockwork. If so, calculate the new flexibility value and return to step 6).
- 9) Repeat steps 4) to 8) until the shear forces at each level remain approximately constant from one iteration to the next.

4.1.1 Net Frame Deformation

The movement of the frame at the top of each level (relative to the base of each level) can be determined by combining the unrestrained movements in the level and applying the appropriate building loads. If the subscript l indicates the building level under consideration, and i the time of interest, then the following terms can be defined as follows;

$$1) \Delta CONCL_{l,i} = (STHT_l) [\epsilon_{th_{CL}}(t_i) + \epsilon_{sh_{CL}}(t_i) + \epsilon_{cr_{CL}}(t_i)]_l \quad (4.1)$$

= unrestrained thermal, shrinkage, and creep movements of the concrete from the time of creation of the column at level l to the time of interest, t_i , in mm

$$2) \Delta BL_{l,i} = (STHT_l) [\epsilon_{th_{BL}}(t_i) + \epsilon_{sh_{BL}}(t_i) + \epsilon_{cr_{BL}}(t_i)]_l \quad (4.2)$$

= unrestrained thermal, shrinkage, and creep movements of the blockwork from the time of placement of the blockwork at level l to the time of interest, t_i , in mm

$$3) \Delta STCL_{l,i} = (STHT_l) [\epsilon_{st_{CL}}(t_i)]_l \quad (4.3)$$

= unrestrained thermal movement in the column steel (reinforcement or structural) at level l at the time of interest, t_i , in mm

$$4) \Delta STBL_{l,i} = (STHT_l) [\epsilon_{st_{BL}}(t_i)]_l \quad (4.4)$$

= unrestrained thermal movement in the blockwork reinforcement at level l at the time of interest, t_i , in mm

where $STHT_l$ is the storey height of level l in mm . Note that the creep strain at the time t_i is dependent on the previous stress history only.

In some reinforced concrete or steel frame structures, the blockwork is installed with a small gap left between its upper surface and the supported slab. Until this gap is closed, the blockwork takes no part in the net vertical frame deformations. To determine this 'closure' condition, it is necessary to determine the net movements of the column and blockwork separately and compare the differential to the effective gap, as follows;

$$\delta COL_{l,i} = \left\{ \frac{(STHT_l) \{ PC_{l,i} - (\sigma CN_{l,i-1})(ACN_l) \} + \{ (E_{st})(ASCN_l)(\Delta STCL_{l,i}) \} + \{ (EC_l(t_i))(ACN_l)[\delta COL_{l,i-1} + \Delta CONCL_{l,i} - \Delta CONCL_{l,i-1}] \}}{[E_{st} \cdot ASCN_l] + [EC_l(t_i) \cdot ACN_l]} \right\} \quad (4.5)$$

which, for a steel column, simplifies to;

$$\delta COL_{l,i} = \Delta STCL_{l,i} + \frac{(STHT_l)(PC_{l,i})}{(E_{st})(A_{st})} \quad (4.6)$$

where,

$\delta COL_{l,i}$ = net displacement between the top and bottom of the column at level l due to building loads and column strains since its creation, in mm

$PC_{l,i}$ = building loads applied to the top of level l at the time of interest, in Newtons. Includes masonry loads above level l , if they are transmitted to the frame, as well as a varying portion of the live load (depending on the severity of the situation investigated). Compressive loads are negative.

$\sigma CN_{l,i-1}$ = stress in the concrete during the previous interval due to building loads and column strains only (ie., not including shelf angle shear forces), in MPa. See Equation (4.7)

ACN_l = cross-sectional area of the concrete in the column at level l (mm²)

$ASCN_l$ = cross-sectional area of the reinforcing steel in the column at level l (mm²)

A_{st} = cross-sectional area of the steel column at level l (mm²)

E_{st} = modulus of elasticity of the steel in MPa

$EC_l(t_i)$ = modulus of elasticity of the concrete at level l at the time of interest, t_i , in MPa

The stress memory factor, $\sigma CN_{l,i}$, is necessary to include the effect of changing concrete stiffness with time. It is based on the assumption that the elastic strain under constant load will not change even though the stiffness is changing. Essentially Equation (4.5) applies the new concrete stiffness only to the latest incremental stress and adds the resulting elastic strain to the previous 'remembered' strain in the column. This principle is applied to all of the structural elements whose stiffness changes with time. The stress memory factor, $\sigma CN_{l,i}$, can be calculated as follows;

$$\sigma CN_{l,i} = \sigma CN_{l,i-1} + \left(\frac{EC_l(t_i)}{STHT_l} \right) (\delta COL_{l,i} - \delta COL_{l,i-1} - \Delta CONCL_{l,i} + \Delta CONCL_{l,i-1}) \quad (4.7)$$

Note that if the concrete elastic modulus was constant from one time interval to the next, Equation (4.5) would simplify to;

$$\delta COL_{l,i} = \frac{(STHT_l \cdot PC_{l,i}) + (\Delta STCL_{l,i} \cdot E_{st} \cdot ASCN_l) + (\Delta CONCL_{l,i} \cdot EC_l \cdot ACN_l)}{(E_{st} \cdot ASCN_l) + (EC_l \cdot ACN_l)} \quad (4.8)$$

Since the column movements are in relation to its position at the time of its creation, the gap left above the blockwork must be modified if the blockwork is placed after the column is created. Thus, the modified gap takes on the value that it would have had if the blockwork were placed at the same time as the column.

$$GAPBL_l = GAPBL'_l - \delta COL_{l,j} \quad (4.9)$$

where,

$GAPBL_l$ = effective gap left above the blockwork as seen by the frame, in mm. Positive magnitude only

$GAPBL'_l$ = actual gap left above the blockwork as seen by the mason, in mm. Positive magnitude only

$\delta COL_{l,j}$ = column movement as calculated by Equation (4.5) or (4.6) at the time of placement of the blockwork, in mm. Note, contraction of the column is indicated by a negative value.

To calculate the net deformation of the blockwork, it is first assumed not to be loaded by the frame;

$$\delta BL_{l,i} = \frac{\left\{ [\delta BL_{l,i-1} + \Delta BL_{l,i} - \Delta BL_{l,i-1}] (EMBL_l(t_i) \cdot ABL_l) + (\Delta STBL_{l,i} \cdot E_{st} \cdot ASBL_l) - (\sigma_{BL_{l,i-1}} \cdot ABL_l \cdot STHT_l) \right\}}{(E_{st} \cdot ASBL_l) + (EMBL_l(t_i) \cdot ABL_l)} \quad (4.10)$$

where,

$\delta BL_{l,i}$ = net displacement between the top and bottom of the blockwork at level l , at the time of interest, in mm (due to internal unrestrained movements of steel and blockwork only)

$\sigma_{BL_{l,i-1}}$ = stress present in the blockwork concrete during the previous time interval in MPa (due to internal strains only)

ABL_l = net cross-sectional area of the concrete in the blockwork at level l in mm²

$ASBL_l$ = cross-sectional area of the reinforcement in the blockwork at level l in mm²

$EMBL_l(t_i)$ = modulus of elasticity of the blockwork at level l at the time of interest, t_i , in MPa

The stress memory factor, $\sigma_{BL_{l,i}}$, is required to take into account the varying elastic modulus of the blockwork, and can be found as follows;

$$\sigma_{BL_{l,i}} = \sigma_{BL_{l,i-1}} + \left(\frac{EMBL_l(t_i)}{STHT_l} \right) (\delta BL_{l,i} - \delta BL_{l,i-1} - \Delta BL_{l,i} + \Delta BL_{l,i-1}) \quad (4.11)$$

Note that $\delta BL_{l,i} = \Delta BL_{l,i}$ and $\sigma B_{l,i} = 0$ if $ASBL_l = 0$.

Now the movement of the column and blockwork can be compared to determine if the upper slab comes into contact with the blockwork, ie;

$$\text{if } \delta BL_{l,i} - GAPBL_l - \delta COL_{l,i} \leq 0,$$

then the gap has not closed and the net frame deformation at level l can be found to be;

$$\delta FRAME_{l,i} = \delta COL_{l,i} \quad (4.12)$$

On the other hand, if $\delta BL_{l,i} - GAPBL_l - \delta COL_{l,i} > 0$, then the gap has closed and the net frame deformation becomes;

$$\delta FRAME_{l,i} = \left\{ \begin{array}{l} (PC_{l,i} \cdot STHT_l) + (\Delta STCL_{l,i} \cdot E_{st} \cdot ASCN_l) + [E_{st} \cdot ASBL_l (\Delta STBL_{l,i} - GAPBL_l)] \\ + EC_l(t_i) \cdot ACN_l (\delta FRAME_{l,i-1} + \Delta CONCL_{l,i} - \Delta CONCL_{l,i-1}) \\ + EMBL_l(t_i) \cdot ABL_l [\Delta BL_{l,i} - GAPBL_l + a_l \cdot (\delta FRAME_{l,i-1} + GAPBL_l - \Delta BL_{l,i-1})] \\ - (\sigma B_{l,i-1} \cdot ABL_l \cdot STHT_l) - (\sigma CN_{l,i-1} \cdot ACN_l \cdot STHT_l) \\ \hline (ASCN_l \cdot E_{st}) + (ASBL_l \cdot E_{st}) + (ABL_l \cdot EMBL_l(t_i)) + (ACN_l \cdot EC_l(t_i)) \end{array} \right\} \quad (4.13)$$

where,

$\delta FRAME_{l,i}$ = net deformation between the top and bottom of the frame at level l at the time interest, t_i , since the time of the column placement, in mm (as if the brick veneer were not present)

$\sigma CN_{l,i-1}$ = stress memory factor for stress in the concrete due to the net frame deformation during the previous time interval (MPa)

$\sigma B_{l,i-1}$ = stress memory factor for stress in the blockwork due to the net frame deformation during the previous time interval in MPa (equals zero if the gap above the block had not closed during the previous interval)

$a_l = 1.0$ if the blockwork gap was closed in the previous interval
 $= 0.0$ otherwise

The stress memory factors $\sigma CN_{l,i}$ and $\sigma B_{l,i}$ can be found as follows;

$$\sigma CN_{l,i} = \sigma CN_{l,i-1} + \left(\frac{EC_l(t_i)}{STHT_l} \right) (\delta FRAME_{l,i} - \delta FRAME_{l,i-1} - \Delta CONCL_{l,i} + \Delta CONCL_{l,i-1}) \quad (4.14)$$

$$\sigma_{B2l,i} = \sigma_{B2l,i-1} + \left(\frac{EMBL_l(t_i)}{STHT_l} \right) \left\{ \begin{aligned} &\delta_{FRAMEl,i} + GAP_{BLl} - \Delta_{BLl,i} \\ &- \alpha_l \cdot (\delta_{FRAMEl,i-1} + GAP_{BLl} - \Delta_{BLl,i-1}) \end{aligned} \right\} \quad (4.15)$$

For a steel frame, Equation (4.13) reduces to;

$$\delta_{FRAMEl,i} = \left\{ \begin{aligned} & (PC_{l,i} \cdot STHT_l) + (\Delta_{STCLl,i} \cdot E_{st} \cdot AS_l) - (\sigma_{B2l,i-1} \cdot ABL_l \cdot STHT_l) \\ & + EMBL_l(t_i) \cdot ABL_l [\Delta_{BLl,i} - GAP_{BLl} + \alpha_l \cdot (\delta_{FRAMEl,i-1} + GAP_{BLl} - \Delta_{BLl,i-1})] \\ & + E_{st}' \cdot ASBL_l (\Delta_{STBLl,i} - GAP_{BLl}) \end{aligned} \right\} \frac{1}{(AS_l \cdot E_{st}) + (ASBL_l \cdot E_{st}) + (ABL_l \cdot EMBL_l(t_i))} \quad (4.16)$$

where $\sigma_{B2l,i}$ is calculated according to Equation (4.15).

If the building is a loadbearing masonry structure, then the blockwork is the supporting element and the gap is nonexistent. In this case the frame deformation can be calculated directly, ignoring Equations (4.5) to (4.16), as follows;

$$\delta_{FRAMEl,i} = \left\{ \begin{aligned} & STHT_l [PC_{l,i} - (ABL_l \cdot \sigma_{B2l,i-1})] + (\Delta_{STBLl,i} \cdot E_{st} \cdot ASBL_l) \\ & + ABL_l \cdot EMBL_l(t_i) (\delta_{FRAMEl,i-1} + \Delta_{BLl,i} - \Delta_{BLl,i-1}) \end{aligned} \right\} \frac{1}{(ASBL_l \cdot E_{st}) + (ABL_l \cdot EMBL_l(t_i))} \quad (4.17)$$

$$\sigma_{B2l,i} = \sigma_{B2l,i-1} + \left(\frac{EMBL_l(t_i)}{STHT_l} \right) (\delta_{FRAMEl,i} - \delta_{FRAMEl,i-1} - \Delta_{BLl,i} + \Delta_{BLl,i-1}) \quad (4.18)$$

If the deformation at the time of placement of the upper shelf angle is defined as the datum from which further deformation results in movement relative to the veneer, then we can define another term to record this datum, ie.;

let δ_{FRSA_l} = the value of δ_{FRAMEl} at the time of placement of the upper shelf angle at level l in mm

4.1.2 Net Veneer Deformation

The net unrestrained deformation of the veneer is much simpler to determine than that of the frame. The self-weight strain is assumed to be negligible within an individual panel, and therefore no elastic strain needs to be considered at this point. The creep strain can be considered to be strictly a function of the previous stress history in the veneer, and is therefore independent of the, as yet unknown, present stress level.

$$\text{Define } \Delta VEN_{l,i} = STH \hat{T}_l \cdot [\epsilon_{th_{un}}(t_i) + \epsilon_{mo_{un}}(t_i) + \epsilon_{fr_{un}}(t_i) + \epsilon_{cr_{un}}(t_i) + \epsilon_{co}(t_i)]_l \quad (4.19)$$

= unrestrained thermal, moisture, freezing, creep, and corrosion
movement in the veneer between the top and bottom of level l ,
since placement, to the time of interest, t_i , in mm

Since we are primarily interested in relative movements which occur after the placement of the upper shelf angle at level l , we can further define;

ΔVEN_{SA_l} = the value of ΔVEN_l at the time of placement of the upper shelf
angle at level l in mm

4.1.3 Flexibility of the Frame and Veneer

The flexibility of the frame, as seen by the veneer, can be found at the time of interest as follows;

$$F_{FRl} = \frac{STHT_l}{(ACN_l \cdot EC_l(t_i)) + (ASCN_l \cdot E_{st}) + a_l \cdot [(ABL_l \cdot EMBL_l(t_i)) + (ASBL_l \cdot E_{st})]} \quad (4.20)$$

where,

F_{FRl} = flexibility of the frame at level l in mm/N

$a_l = 1.0$ if the gap above the blockwork has closed
 $= 0.0$ otherwise

If the structure has a steel frame, set $ACN_l = 0$ and use AS_l in place of $ASCN_l$. For a loadbearing masonry structure, set the following;

$$ACN_l = 0.0$$

$$ASCN_l = 0.0$$

$$a_l = 1.0$$

The flexibility of the brick veneer (F_{BRl}) can be found, at the time of interest, as follows;

$$F_{BRl} = \frac{STHT_l}{ABR_l \cdot EMBR_l(t_i)} \quad (mm/N) \quad (4.21)$$

where,

ABR_l = the cross-sectional area of the brick veneer in mm^2 ; at level l

$EMBR_l(t_i)$ = the elastic modulus of the brick veneer at level l at the time of interest, t_i , in MPa

4.1.4 Shelf Angle Properties

The stiffness of the shelf angle system is dependent mainly on the shelf angle size and on the type of connection between the angle and the frame. Yura (119) presents a thorough, but complex method of determining the shear stiffness (resistance to vertical displacement of the outer toe of the angle) and the yield point of a shelf angle, arriving at a trilinear relationship between the shear force transmitted across the shelf angle system and the deformation in the system (see Figure 4.2). For the purposes of this report, a simplified bilinear relationship is assumed to be sufficient, as shown in Figure 4.3. The designer must either estimate (see Chapter 7, Tables 7.2 and 7.3), or derive from test values, the following two parameters;

K_{SA} = shear stiffness of the shelf angle system in either direction, i.e. up or down, in units of N/mm per m length of shelf angle

V_{YSA} = force required, in either direction, to result in yielding of the shelf angle system, in units of N/m length of shelf angle

The flexibility of the shelf angle system, for the wall or angle length considered, can now be determined to be;

$$F_{SA} = \frac{1}{TRIB_{br} \cdot K_{SA}} \quad (mm/N) \quad (4.22)$$

where $TRIB_{br}$ is the tributary length of the veneer measured parallel to the building face in m (it has been assumed in this analysis that the shelf angle length is equal to $TRIB_{br}$).

If the rest position of the shelf angle, after the supported brick panel has been placed, is defined as the datum about which further deformations are measured, then the following two parameters must be adjusted;

- 1) Since the expansion gap, left by the mason below the shelf angle, is set prior to placement of the upper brick panel, the gap will narrow slightly due to the weight of the upper brick panel. The final 'effective' gap can be found as follows;

$$GAP_{BRl} = GAP_{BRl}' - (F_{SA} \cdot W_{BRl}) \quad (mm) \quad (4.23)$$

where,

GAP_{BRl} = effective expansion gap width left below the shelf angle at the base of level l in mm

GAP_{BRl}' = specified expansion gap width left below the shelf angle at the base of level l , as seen by the mason, in mm

W_{BRl} = weight of the entire brick panel, between shelf angles, at level l in Newtons

- 2) If this deformed position is taken to be the 'rest' position of the shelf angle, the force required to yield the shelf angle system is now different in the upwards and downwards directions. If V_{YSAU_l} is defined as the force required to yield the shelf angle at the base of level l when the outside tip is pushed upwards, and V_{YSAL_l} as the force required to yield the shelf angle at the base of level l when the outside tip is pushed downwards, in Newtons, then;

$$V_{YSAU_l} = (V_{YSA} \cdot TRIB_{br}) + W_{BRl} \quad (4.24)$$

$$V_{YSAL_l} = (V_{YSA} \cdot TRIB_{br}) - W_{BRl} \quad (4.25)$$

Figure 4.2 Shear-displacement relationship for shelf angles (from Yura, 119).

Figure 4.3 Simplified shear-displacement relationship for shelf angles.

4.1.5 Shelf Angle Shear Forces

In this section, the actual iterative analysis will be described step by step. Working from the bottom level at which the veneer is present, the shear forces in each of the shelf angles will be determined and used to calculate the cumulative force transmitted through the veneer to the lower levels. The variables used in this analysis are described as follows;

- 1) $SREST_l$ is the 'at rest' elevation of the toe of the shelf angle at the top of level l in mm.
- 2) $SPOS_l$ is the actual elevation of the shelf angle toe at the top of level l in mm.
- 3) $BRPOS_l$ is the unrestrained elevation of the upper surface of the brick panel at level l in mm.
- 4) VSA_l is the shear force (Newtons) induced in the shelf angle at the top of level l (positive if exerting an upwards force on the upper brick panel and negative if exerting a downwards force on the underlying brick panel).
- 5) $PVEN_l$ is the cumulative force acting downwards on the shelf angle at the top of level l , in Newtons.
- 6) $ELBR_{l,i}$ is the elastic movement in the brick panel at level l due to the shelf angle forces acting on it at the beginning of interval i , in mm. This is a 'memory' factor to allow the consideration of the varying elastic modulus of the brick veneer.
- 7) $ELFR_{l,i}$ is the elastic movement in the frame at level l due to the shelf angle forces acting on it at the beginning of interval i in mm. This is a 'memory' factor for the frame for shelf angle loading - not to be confused with the memory factors defined in Section 4.1.1 for building loads.
- 8) $V2_l$ is the value of the cumulative compressive force acting on the veneer at level l during the previous time interval in Newtons (ie. $V2_l = PVEN_{l,i-1}$).

The 'at rest' elevation of the shelf angle toe at the top of level l can be found to be;

$$SREST_l = SREST_{l-1} + \Delta VENSAl + GAPBR_{l+1} + (\delta FRAME_{l,i} - \delta FRSA_l) \quad (4.26)$$

Similarly, the 'at rest' elevation of the upper surface of the brick panel at level l will be;

$$BRPOS_l = SREST_{l-1} + \Delta VEN_{l,i} \quad (4.27)$$

If the elevation of the shelf angle, when deformed by the cumulative shear force transmitted from above, is below that of the upper surface of the underlying brick panel, ie;

$$\{SREST_l + (FSA \cdot PVEN_l)\} < BRPOS_l$$

then the upper shelf angle comes into contact with the underlying brick panel and the shear force developed in the shelf angle will be;

$$VSA_l = \left\{ \frac{SREST_l - SREST_{l-1} - \Delta VEN_{l,i} + ELFR_{l,i-1} - ELBR_{l,i-1}}{FBR_l + FFR_l + 2FSA} \right\} \quad (4.28)$$

If the next lower shelf angle is supported, i.e. already in contact with the underlying brick or foundation ($SPOS_{l-1} > SREST_{l-1}$), then Equation (4.28) simplifies to;

$$V_{SA_l} = \left\{ \frac{SREST_l - SREST_{l-1} - \Delta VEN_{l,1} + ELFR_{l,1-1} - ELBR_{l,1-1}}{FBR_l + FFR_l + FSA} \right\} \quad (4.29)$$

If neither the top or bottom shelf angle has yielded, then the actual elevation of the upper shelf angle can be found to be;

$$SPOS_l = SREST_l - (FSA \cdot V_{SA_l}) \quad (4.30)$$

However, if the upper shelf angle yields while the bottom shelf angle is supported, then V_{SA_l} must be limited to the appropriate yield value (either V_{YSAU} or V_{YSAL} depending on the direction) and the actual elevation of the upper shelf must be determined as follows;

$$SPOS_l = SPOS_{l-1} + \Delta VEN_{l,1} + ELBR_{l,1} - ELFR_{l,1} \quad (4.31)$$

If the bottom shelf angle is unsupported and yields in the downwards direction, then the shear force developed in the upper shelf angle is limited to;

$$V_{SA_l} = -PVEN_l - V_{YSAL_l} \quad (4.32)$$

and the elevation of the upper shelf angle can be calculated using Equation (4.30).

The elastic movement 'memory' factors can now be determined as follows;

$$ELBR_{l,1} = ELBR_{l,1-1} + (FBR_l)(PVEN_l + V_{SA_l} - V2_l) \quad (4.33)$$

$$ELFR_{l,1} = ELFR_{l,1-1} - (FFR_l)(PVEN_l + V_{SA_l} - V2_l) \quad (4.34)$$

If both of the shelf angles yield in the downwards direction, then, although the shear forces will be known, the rest elevation of the shelf angles must be estimated. Under these conditions, the cumulative force transmitted from above cannot be greater than the sum of the yield forces in the two shelf angles considered. If 'DP' is defined as the displacement (beyond the yield displacement), in mm, required to reduce the cumulative force ' $PVEN_l$ ' to the yield force, then;

$$DP \approx \frac{(FSA)(PVEN_l + V_{YSAL_l} + V_{YSAL_{l+1}})}{(\text{number of shelf angles above level } l)} \quad (4.35)$$

Note that $PVEN$ will be a negative value while V_{YSAL} is a positive magnitude, making DP negative

for downward movement. Now;

$$V_{SA_l} = V_{YSAL_{l+1}} \quad (4.36)$$

$$SPOS_l = SREST_l - (V_{SA_l} \cdot F_{SA}) + DP \quad (4.37)$$

$$SPOS_{l-1} = SPOS_l - \Delta VEN_{l,i} - ELBR_{l,i} + ELFR_{l,i} \quad (4.38)$$

$$ELBR_{l,i} = ELBR_{l,i-1} - (FBR_l)(V_{YSAL_l} + V2_l) \quad (4.39)$$

$$ELFR_{l,i} = ELFR_{l,i-1} + (FFR_l)(V_{YSAL_l} + V2_l) \quad (4.40)$$

If Equation (4.38) gives a shelf angle elevation lower than the underlying brick or foundation, then the lower shelf angle does in fact become 'supported' and the true displacement can be calculated accordingly using Equations (4.31) to (4.34) with $SPOS_{l-1} = BRPOS_{l-1}$. Note the assumptions that the underlying brick panel is a fixed support and that overlying panels are entirely free to move. These assumptions are actually only applied within the framework of a single iteration, the final interaction being expressed in the cumulative shear force term, $PVEN$, which, after all the shear forces have been calculated, can be found to be;

$$PVEN_{l,j} = PVEN_{l,j-1} - (DMP_l) \left\{ PVEN_{l,j-1} - \sum_{n=l+1}^N V_{SA_n} \right\} \quad (4.41)$$

where the subscript j indicates the number of iterations which have taken place at this point, and N is the number of levels in the building. The term (DMP_l) is a damping factor designed to prevent the iterative analysis from becoming unstable. For the purposes of this report, it has been found empirically that satisfactory results are obtained if the damping factor is set as follows;

$$DMP_l = \left(\frac{1}{(N-l)+1} \right)^{0.7}$$

To ensure that the cumulative force transmitted through the veneer becomes equal to zero immediately below an unsupported shelf angle, the shear force in that angle must be set by statics to;

$$V_{SA_l} = -PVEN_l \quad (4.42)$$

Before proceeding to the next iteration, the frame stiffness must be checked to see if it has changed due to liftoff from the blockwork (it is assumed that the blockwork cannot go into tension). This step is only applicable to steel or concrete frames when the infill block masonry is present.

First the deformation of the column, as if the blockwork were not present, due to the shelf angle forces must be calculated as follows;

$$\Delta = \frac{ELCL_{l,i-1} - (STHT_l)(PVEN_l + V_{SA_l} - V2_l)}{(ACN_l \cdot EC_l(t)) + (ASCN_l \cdot E_{st})} \quad \text{for concrete frames} \quad (4.43)$$

The term $ELCL_i$, is the elastic movement memory factor for the column alone due to shelf angle forces and is equal to the value of Δ calculated during the previous time interval.

For a steel frame, Equation (4.43) reduces to;

$$\Delta = \frac{-STHT_i(PVEN_i + VSA_i)}{AS_i \cdot E_{st}} \quad \text{for steel frames} \quad (4.44)$$

Now if $\{\delta BL_{i,i} - GAPBL_i - \delta COL_{i,i} - \Delta\} > 0$ then the block is in contact with the frame. If this condition represents a change from the original assumption, then the frame stiffness must be recalculated according to Section 4.1.3 and the iteration repeated.

In this fashion the iterative process can be continued until the shear forces calculated in one step are sufficiently close to those calculated in the next. In the program developed for this report, the maximum acceptable difference in the veneer axial stress, calculated in subsequent iterations, is 0.002 MPa. Although this appears to be very accurate, it must be remembered that the error is cumulative from one floor to the next. Thus for a 25 storey structure (with 25 shelf angles) the actual maximum error is about 0.05 MPa.

4.2 Final Stresses

Having determined the shear force in each of the shelf angles, the final stress in each of the building elements can be evaluated for this particular time interval. The stress in the veneer can be found quite simply from statics;

$$\sigma_{BRl,i} = \frac{V_{SA_l} + P_{VEN_l}}{A_{BRl}} \quad (\text{MPa}) \quad (4.45)$$

where,

$\sigma_{BRl,i}$ = the average axial stress in the brick veneer panel at level l during the time interval i , in MPa

A_{BRl} = the net cross-sectional area of the veneer at level l in mm^2

Because of its composite nature, the stress in the elements of the concrete column must be found somewhat less directly, making use of the stress memory factors. If the blockwork is not acting with the frame (ie. liftoff has occurred or the gap left above the blockwork has not yet closed), then one of the concrete stress memory factors, $\sigma_{CN1l,i}$, has already been defined in Equation (4.7). The effect of the shelf angle shear forces can be included in a third stress memory factor (the second is used when the blockwork is acting with the frame), as follows;

$$\sigma_{CN3l,i} = \sigma_{CN3l,i-1} - \frac{E_{Cl}(t_i) \cdot (P_{VEN_l} + V_{SA_l} - V_{2l})}{(AC_{Nl} \cdot E_{Cl}(t_i)) + (ASC_{Nl} \cdot E_{st})} \quad (\text{MPa}) \quad (4.46)$$

Similarly, a stress memory factor for the reinforcement can be defined as follows;

$$\sigma_{S1l,i} = \sigma_{S1l,i-1} - \frac{E_{st} \cdot (P_{VEN_l} + V_{SA_l} - V_{2l})}{(AC_{Nl} \cdot E_{Cl}(t_i)) + (ASC_{Nl} \cdot E_{st})} \quad (\text{MPa}) \quad (4.47)$$

Thus, for the frame acting independently of the blockwork, the concrete and reinforcement stresses, $\sigma_{CONCl,i}$ and $\sigma_{SCLl,i}$ respectively, in the column, are;

$$\sigma_{CONCl,i} = \sigma_{CN1l,i} + \sigma_{CN3l,i} \quad (\text{MPa}) \quad (4.48)$$

$$\sigma_{SCLl,i} = \sigma_{S1l,i} + \frac{E_{st} \cdot (\delta_{COLl,i} - \Delta_{STCLl,i})}{STHT_l} \quad (\text{MPa}) \quad (4.49)$$

In a steel frame, acting independently of the blockwork, the column stress can be found simply to be;

$$\sigma_{SCLl,i} = \frac{P_{Cl,i} - (P_{VEN_l} + V_{SA_l})}{A_{Sl}} \quad (\text{MPa}) \quad (4.50)$$

Blockwork Acting with the Frame

When the blockwork is acting with the frame, i.e. is carrying some load, several new stress memory factors must be determined;

1) For the concrete,

$$\sigma_{CN4l,i} = \sigma_{CN4l,i-1} - \left(\frac{EC_l(t_i)}{Z} \right) (P_{VENl} + V_{SA_l} - V_{2l}) \quad (MPa) \quad (4.51)$$

2) For the column reinforcement,

$$\sigma_{S2l,i} = \sigma_{S2l,i-1} - \left(\frac{E_{st}}{Z} \right) (P_{VENl} + V_{SA_l} - V_{2l}) \quad (MPa) \quad (4.52)$$

3) For the blockwork,

$$\sigma_{B3l,i} = \sigma_{B3l,i-1} - \left(\frac{EMBL_l(t_i)}{Z} \right) (P_{VENl} + V_{SA_l} - V_{2l}) \quad (MPa) \quad (4.53)$$

4) For the blockwork reinforcement,

$$\sigma_{SB1l,i} = \sigma_{SB1l,i-1} - \left(\frac{E_{st}}{Z} \right) (P_{VENl} + V_{SA_l} - V_{2l}) \quad (MPa) \quad (4.54)$$

where,

$$Z = (ACN_l \cdot EC_l(t_i)) + (ASCN_l \cdot E_{st}) + (ABL_l \cdot EMBL_l(t_i)) + (ASBL_l \cdot E_{st})$$

The concrete stress can now be calculated to be;

$$\sigma_{CONC_{l,i}} = \sigma_{CN2_{l,i}} + \sigma_{CN4_{l,i}} \quad (MPa) \quad (4.55)$$

where the stress factor $\sigma_{CN2_{l,i}}$ was defined in Equation (4.14).

The column reinforcement stress will be;

$$\sigma_{SCL_{l,i}} = \sigma_{S2_{l,i}} + \frac{E_{st} \cdot (\delta_{FRAME_{l,i}} - \Delta_{STCL_{l,i}})}{STHT_l} \quad (MPa) \quad (4.56)$$

The blockwork stress will be;

$$\sigma_{BL_{l,i}} = \sigma_{B2_{l,i}} + \sigma_{B3_{l,i}} \quad (MPa) \quad (4.57)$$

where the stress factor $\sigma_{B2_{l,i}}$ was defined in Equation (4.15).

The blockwork reinforcement stress will be;

$$\sigma_{SBL_{l,i}} = \sigma_{SB1_{l,i}} + \frac{E_{st} \cdot (\delta FRAME_{l,i} + GAPBL_l - \Delta STBL_{l,i})}{STHT_l} \quad (MPa) \quad (4.58)$$

For a steel frame in which the blockwork gap has closed, an additional stress memory factor can be defined;

$$\sigma_{S3_{l,i}} = \sigma_{S3_{l,i-1}} - \frac{E_{st} \cdot (PVEN_l + VSA_l - V2_l)}{(AS_l \cdot E_{st}) + (ABL_l \cdot EMBL_l(t_i)) + (ASBL_l \cdot E_{st})} \quad (MPa) \quad (4.59)$$

and the stress in the steel column can be found as follows;

$$\sigma_{SCL_{l,i}} = \sigma_{S3_{l,i}} + \frac{E_{st} \cdot (\delta FRAME_{l,i} - \Delta STCL_{l,i})}{STHT_l} \quad (MPa) \quad (4.60)$$

In a loadbearing masonry structure, the stress memory factors are similar to those developed previously for blockwork, appearing as follows;

$$\sigma_{B3_{l,i}} = \sigma_{B3_{l,i-1}} - \frac{EMBL_l(t_i) \cdot (PVEN_l + VSA_l - V2_l)}{(ABL_l \cdot EMBL_l(t_i)) + (ASBL_l \cdot E_{st})} \quad (MPa) \quad (4.61)$$

$$\sigma_{SB1_{l,i}} = \sigma_{SB1_{l,i-1}} - \frac{E_{st} \cdot (PVEN_l + VSA_l - V2_l)}{(ABL_l \cdot EMBL_l(t_i)) + (ASBL_l \cdot E_{st})} \quad (MPa) \quad (4.62)$$

The stress in the block and its reinforcement can now be calculated using Equations (4.57) and (4.58) with $GAPBL_l = 0$.

Chapter 5

Computer Program

5.1 Introduction

To enable the designer not only to estimate the long term vertical movements discussed in the previous chapters, but also to arrive at the stresses in each of the building elements, a computer program called VENEER was developed on the basis of the theory presented in Chapter 4. The program calculates movements and stresses at up to eight specified times, during the life of the building, for the following types of structures;

- 1) reinforced concrete frame with or without blockwork infill,
- 2) structural steel frame with or without blockwork infill, and
- 3) loadbearing masonry frame.

Buildings up to 25 storeys in height (including basements) can be handled.

In this chapter, the advantages and limitations of the program will be discussed, along with any simplifying assumptions which were made. As with any computer program, certain rules must be followed to be properly used, and these will be pointed out to the user.

5.2 Limitations

The limitations placed on the user by the program are generally of two types;

- 1) those required to make the program more manageable in terms of memory space, and
- 2) those due to the shortcomings of the program itself.

Fortunately the limitations arising from the second category are not many. As noted previously, one of these limitations is that the program will only handle reinforced concrete, steel, and loadbearing masonry structures (not timber). However, this is not a serious limitation since most highrise structures fall into one of these categories. Perhaps a more serious limitation is that the structural materials must be from the following list;

- 1) Clay brick
- 2) Concrete block (reinforced if desired)
- 3) Reinforced concrete

4) Structural steel

Although dummy subroutines have been included in the program for loadbearing clay block (reinforced if desired) and concrete brick (for use as a veneer), they have yet to be written, and so cannot be used at this time. Note that in the data input procedure given in Appendix E, these materials are provided as options to the user, but their use will result in an error message.

The limitations of the program required to minimize the used memory space are much more numerous. They will be discussed individually, in the following list, in the order that they would be encountered by the user when entering the building data;

- 1) The total number of building levels, including basements, must be less than or equal to 25. If the user wishes to consider a taller structure, he can do so by considering one level in the model to be equivalent to two levels in the real structure. This involves doubling the floor loads and shelf angle parameters, and making the appropriate adjustments to the storey heights and to the times of placement of the various elements.
- 2) The user is limited to one set of shelf angle parameters (stiffness and yield strength) which are assumed to apply to all shelf angles provided. Note that in a loadbearing masonry structure, the ties are assumed to be lumped at each floor level and are considered to act as shelf angles.
- 3) A maximum of four different wall cross-sections can be considered for reinforced concrete or steel structures. Within each panel the wall cross-section must be specified at both the column location and at a mid-bay location. In addition, a different set of cross-sections can be specified for one other group of levels, making four cross-sections in total. Since there is no column in a loadbearing masonry structure, the maximum number of different cross-sections is two, each one typical at two different groups of levels (ie., from level 1 to level 4 and from level 5 to the roof).
- 4) In each cross-section specified, the only layer type which can be repeated is the air gap (to a maximum of two air cavities in a cross-section). If a double layer of any other material actually exists in a wall, simply consider it as a single layer and make the appropriate change in its thickness.
- 5) Only one set of shelf angle corrosion parameters (ie., volume change ratio, surface thickness corroded, and the age of the shelf angle at the time of measurement) can be specified. These are assumed to apply to all the shelf angles provided and so it is important to use average or representative values.
- 6) Many of the material properties are considered to be constant for each material, wherever it is found. They are as follows;
 - a) tributary length of the brick veneer, blockwork, insulation, wall finish, and the air cavity(s),
 - b) density of each material,
 - c) specific heat of each material,

- d) thermal conductivity of each material,
 - e) thickness of the brick, insulation, wall finish, and air cavity(s),
 - f) emissivity and absorptivity of each material,
 - g) coefficient of thermal expansion of each material,
 - h) freezing expansion parameters of clay brick,
 - i) moisture expansion or shrinkage parameters of clay brick, concrete block, and concrete.
- 7) A maximum of eight 'times of interest' may be specified. At these times the 'extreme' conditions discussed in Chapter 4 are applied to the building and the resulting stresses computed and printed out.
- 8) The program will consider a maximum of 50 time intervals altogether. Since a new time interval is generated at each time of placement of each of the structural elements (brick veneer, blockwork, column, and the shelf angle) as well as at the times the shoring at each level is removed, it would not be difficult to exceed this limit in a taller building. To avoid this problem, it may be necessary to adjust the time of placement of certain elements so that they coincide with the time of placement of other elements. For example, if the veneer at one level was placed at the same time as the blockwork at some level, only one time interval would be generated for this occurrence. Beyond the construction phase, the time intervals exponentially increase in length (with certain values corresponding to the specified times of interest) and, in most cases, do not contribute more than 15 time intervals to the analysis.

5.3 Assumptions

A great many assumptions were made in the program to simplify the analysis. In this section the steps taken by the program will be described, along with the assumptions that were made at that point, in the order that they would be encountered during the execution of the program. Figure 5.1 shows the program flow chart in which each box represents a subroutine. Only the major subroutines names will be indicated in the following discussion, however it should be realized that some, such as RDATA and SHEAR, incorporate a large number of smaller subroutines. The interested user should obtain a copy of the program itself for individual descriptions of the smaller subroutines.

Figure 5.1 Flow chart of the program VENEER.

5.3.1 Data Input (RDATA)

The data entry procedure to be followed by the user is described fully in Appendix E. In subroutine RDATA, however, many preliminary calculations are performed to modify the data into a form usable by the remainder of the program (immediately upon receipt of the data values involved). The preliminary calculations often involve certain assumptions and will be listed as follows in the order they would appear in the execution of the program;

- 1) From the vertical and horizontal shading angles, the surface to sky view factors for each level are calculated assuming that the obstructions defined by the horizontal shading angles are infinitely tall (as seen by the level under consideration only)
- 2) The cross-sectional area of the veneer is calculated from its thickness and length assuming that the brick units are 85% solid. For different values of 'percent solid', the user should adjust the thickness of the veneer. For instance, for a solid brick unit, the thickness should be increased by 18%
- 3) If the thermal conductivity of the blockwork is unspecified, the program calculates it using an average block thickness equal to the sum of the true block thicknesses at each level divided by the number of levels.
- 4) The shrinkage of the blockwork is calculated assuming that the additional effect of grout shrinkage is proportional to its relative area. This is based on the assumption that the elastic moduli of the two materials are the same.

5.3.2 Building Timing (RDATA)

In this portion of the program, the time intervals to be used in the analysis are calculated from the time of placement of the various building elements. Beyond the end of construction the time intervals increase in length exponentially (10, 20, 40, 80, 160, 320, 640, 1280 days etc.) and/or correspond to the times of interest specified by the user. Since the load on any of the structural elements is assumed to be constant throughout each time interval, it is important that the beginning of each time interval in the construction phase correspond to either,

- 1) the placement of a massive element (i.e., veneer, blockwork, column, or slab), or
- 2) the placement or removal of an element resulting in a transfer of load (i.e., the removal of the shoring results in a transfer of load to the column and the placement of the shelf angle results in a potential transfer of load to the veneer).

Note that the live load (or portion thereof) is assumed to act on a level immediately upon placement of the slab.

5.3.3 Load History (LDHIST)

The program now calculates the load acting at the top of each level at the beginning of each time interval. The assumptions involved in this calculation are as follows;

- 1) If shoring is present at the time and level considered, the load is assumed to be transmitted directly to the underlying level. Thus the load acting on the supporting elements at this level would be zero at this time.
- 2) Veneer loads are considered to be entirely transmitted to the frame through the shelf angles (but not ties) at the shelf angle locations, i.e., the shelf angles are considered to be unsupported even if no expansion joint is provided (see Section 5.3.6).

5.3.4 Initial Temperatures (INIT)

The initial temperatures of each of the structural elements, blockwork, column, and the veneer, are calculated as being equal to the average ambient air temperature at the time of their placement, at each level. If construction occurs during the winter, however, some protection would be supplied to allow the normal curing of the element. For this reason, the initial temperatures are assumed to be not less than -5°C .

5.3.5 Effective Column Thickness (COLBF)

To allow a reasonably accurate determination of the average column temperature using a one-dimensional heat flow analysis, it is necessary to determine the effective projecting thickness of the column (see Chapter 2 and Appendix B). Although the relationships used to determine the effective thickness are based on steady state heat flow, it is further assumed that they are applicable to transient heat flow. There is some question as to whether an effective 'thermal mass' of the column should also be determined for use in the transient heat flow analysis. In the absence of research on the subject, however, it will be assumed that the thermal mass of the effective column is identical to that of the true column (per unit volume).

5.3.6 Final Expansion Gap Under Each Shelf Angle (BRGAP)

When the shelf angle is placed, there should be a small gap left between it and the underlying veneer. The specified thickness of this expansion gap depends upon many design considerations. However, once the overlying brick panel has been placed, the actual gap thickness will be somewhat reduced due to the brick weight deforming the shelf angle. The program calculates the final gap thickness under each shelf angle as if the shelf angle is entirely unsupported. Depending on the mass of the brick veneer, the stiffness of the shelf angle, and the original specified gap thickness, the final gap thickness may even be a negative value. Considering this deformed position of the

shelf angle to be its 'rest' position allows the brick loads to be added directly to the frame loads (through the appropriate shelf angles). Since the weight of the overlying brick panel is now included in the shelf angle position, the stress in the underlying brick panel is strictly dependent on how it deforms the shelf angle (ie., the additional stress due to the self-weight of the overlying brick panels is already included).

5.3.7 Basement Heat Flow (BSFLW)

In this subroutine, the program calculates the heat flow (and thus the element temperatures) through the basement exterior wall sections. The basic assumption made here is that the soil temperature, one metre out from the exterior wall surface, is constant and equal to the mean annual temperature. This assumption is reasonable for depths greater than about one or two metres below the ground surface. A one metre thickness of soil is assumed to act with the wall, having a thermal resistance of $1.98 \text{ m}^2 \cdot ^\circ \text{C/Watt}$. The element temperatures are found using a steady state, one-dimensional heat flow analysis. Note that these temperatures are assumed to be constant throughout the life of the building and so are primarily calculated for interest's sake since they do not lead to differential movements.

5.3.8 Transient Heat Flow Analysis (THERM2)

To take into account the effect of solar radiation on the temperatures, and thus the thermal strains, of the elements of the exterior wall, the program now performs a transient heat flow analysis (described in greater detail in Chapter 2). The analysis actually consists of two parts;

- 1) determination of element temperatures on an hourly basis over a 24 hour period, and
- 2) extraction of the temperature sets which give the maximum and minimum differential movement between the veneer and the frame.

Although the temperature set which gives the minimum differential movement is only necessary for loadbearing masonry structures, it is also extracted for concrete and steel structures.

The assumptions made to simplify both the transient heat flow analysis and the overall program are as follows;

- 1) The temperature values derived from the analysis are assumed to be applicable at any summer time of interest specified by the user.
- 2) The exterior wind speed is considered to be constant over the height of the building and has a value equal to the mean wind speed value derived from the specified nominal wind speed applied with the distribution given by the National Building Code of Canada (71).
- 3) The interior air temperature is assumed to remain constant if the building is air conditioned and the windows are closed. If the windows are open, the interior temperature is assumed to be identical at all times to the exterior ambient temperature. If the building is not air conditioned

and the windows are closed, the interior air temperature is calculated by including a $1 \times 1 \times 3$ metre block of air as one of the elements in the heat flow analysis. Note that the effect of solar heating through the windows is not considered (the windows are assumed to be curtained).

- 4) The heat flow through the wall is assumed to be one dimensional (heat flow lines are parallel).
- 5) The effective column dimension, as determined for the steady state heat flow analysis, is assumed to be also applicable to the case of transient heat flow.
- 6) The thermal resistance of steel is assumed negligible when compared to the thermal resistance of other common building materials.
- 7) For the 24 hour period, over which the transient analysis takes place, the sky is assumed to remain entirely clear.
- 8) If conditions are identical at two or more levels, the heat flow analysis is only performed at the lowest of the identical levels and the results are assumed to be true for all the identical levels.

5.3.9 Steady State Heat Flow Analysis (THERM1)

The steady state heat flow analysis is performed at the beginning of each time interval to determine the wall element temperatures under average ambient conditions. It is also performed to determine temperature profiles under 'extreme' ambient conditions for winter as well as summer times of interest (if the summer transient analysis is not desired).

The major assumptions made in this analysis are as follows;

- 1) The interior air temperature is assumed to be constant during the winter (Oct. 1 to Apr. 1) at the value specified by the user. During the warmer months (Apr. 2 to Sept. 30) the interior air temperature depends on whether the building is air conditioned or the windows open. If the windows are open at the level considered, the interior temperature is assumed to equal the ambient temperature, in which case all the temperatures through the wall are equal. If the windows are closed, however, the interior temperature is assumed equal to the specified interior temperature. In the special case, when the summer 'extreme' ambient temperatures are applied through the steady state analysis, the interior air temperature is assumed to be 5°C above that specified by the user, if the building is not air conditioned and the windows are closed.
- 2) In general, the ambient air temperature is assumed to be equal to the average value expected for the time of year considered (the parameters of the annual average temperature distribution are specified by the user). Since this value is assumed to be constant throughout the time interval considered, for time intervals greater than 180 days in duration the ambient air temperature becomes equal to the mean annual temperature. If the temperature profile of the wall is desired under 'extreme' ambient conditions, the ambient air temperature becomes equal to the appropriate value specified by the user.

- 3) The temperature values derived under 'extreme' ambient conditions are assumed to be true for all the appropriate (ie., summer or winter) times of interest specified by the user.
- 4) The exterior wind speed is considered to vary over the height of the building according to the distribution given by NBC (71) for the nominal windspeed and exposure condition specified by the user.
- 5) The heat flow through the wall is assumed to be one-dimensional.
- 6) The thermal resistance of steel is assumed to be negligible when compared to the thermal resistance of other common building materials.

Subroutine THERM1 is also used in the program to calculate the initial wall element temperatures, just prior to dawn, for use in the transient heat flow analysis.

5.3.10 Shelf Angle Shear Force Analysis (SHEAR)

The program is now ready to calculate the shear forces arising in the shelf angles due to differential movements between the frame and the veneer. The procedure is described in detail in Chapter 4 and basically consists of three parts;

- a) calculate the movements of the veneer and frame as if separate,
- b) calculate the elastic deformation of the veneer and frame when restrained by each other, and
- c) calculate the resultant total stress in each element.

The simplifying assumptions made in this routine can be listed as follows;

- 1) Both the frame and the veneer are assumed to remain fully elastic with some non-linearity introduced due to the changing stiffness with time.
- 2) All forces are considered to act axially through the centroid of each supporting element.
- 3) The shelf angles (or lumped ties) are assumed to be linearly elastic up until yielding takes place, thereafter experiencing no load increase for increasing deformation.
- 4) In steel or reinforced concrete structures, the brick veneer and the blockwork infill are assumed to be unable to develop tensile forces (except for those arising in the blockwork due to block unit - reinforcement interaction).
- 5) All loads, deformations, and stresses are assumed to be constant throughout each time interval.
- 6) The calculated deformations are assumed to reflect the true relative movement between the top and bottom of a structural element at a particular level - no attempt is made to include the self-weight strain within the level itself or differential strains due to varying material properties over the height of the level.

- 7) The principle of superposition is assumed to be applicable to all building elements which experience creep.

5.3.11 Total Differential Movement (LBMDD)

The total differential movement and the total differential thermal movement between the veneer and the frame is now evaluated at the roof level for loadbearing masonry structures only. Because the restraining effect of the ties (to vertical movement) is generally assumed to be negligible, the thermal and total movements at each level can be summed over the height of the building to give the differential movement at the roof level. The differential thermal movement is evaluated on both an annual and a daily basis and represents a cyclic movement.

The exterior wall cross-section appears as shown in Figure 6.2 with the insulation placed on the interior surface of the blockwork. The red-brown clay brick veneer runs uninterrupted from the first floor to the roof and is tied to the blockwork as indicated with horizontal truss ties.

Figure 6.2 Cross-section through exterior wall.

6.2 Observed Exterior Wall Temperatures

Four walls, facing north, south, east, and west, were instrumented with semi-conductor temperature sensors near the roof level, as shown in Figure 6.3. At each wall one sensor was placed on the exterior brick surface and another suspended in the air cavity about one metre down from the top of the wall. Ambient air temperature was obtained using a temperature sensor suspended in the shade to the north of the penthouse.

Figure 6.3 Roof level instrumentation to measure surface and air cavity temperatures.

National Semiconductor LM235 Precision Temperature Sensors were chosen to measure wall temperatures over other temperature sensing systems for two reasons;

- 1) because of their relatively high output voltage change of 10 mV per degree Celsius (linear), inaccuracies arising from electrical 'noise' in long wire lengths are negligible (thermocouple voltage output typically varies by only a few microvolts per degree Celsius and is non-linear), and
- 2) at a cost of about six dollars per sensor, these sensors are considerably less expensive than most thermistors.

The sensors are accurate to within 0.5 °C over their entire range.

Output voltages from the temperature sensors were measured once every two hours, scaled, and the corresponding temperatures printed on a paper strip by a Monitor Labs 9300 Data Logger. The equipment was installed in November 1981 and maintained for about one year. Unfortunately breakdown and servicing of the data logger (which may have been due to dust or source voltage fluctuations in the penthouse) on several occasions led to a loss of about 95 days over the course of the year. However the down times were generally short in duration and did not result in a loss of extreme temperature periods.

Table 6.1 presents a summary of the observed extreme brick surface temperatures for each wall, the time of occurrence, and the corresponding air cavity temperatures.

Table 6.1 Summary of observed extreme wall temperatures.

Wall	East	South	West	North
Max. Brick Surface Temp. ($^{\circ}\text{C}$)	51.5	45.4	55.0	35.7
Time of Occurrence (hrs.)	10:00	15:00	18:00	18:00
Corresponding Air Cavity Temp. ($^{\circ}\text{C}$)	30.8	31.1	42.4	27.5
Min. Brick Surface Temp. ($^{\circ}\text{C}$)	-24.6	-25.2	-27.4	-26.3
Time of Occurrence (hrs.)	6:00	6:00	6:00	6:00
Corresponding Air Cavity Temp. ($^{\circ}\text{C}$)	-20.0	-20.1	-20.0	-21.6

The air cavity temperatures listed in Table 6.1 are those values which occur simultaneously with the extreme brick surface temperatures (not necessarily the maximum or minimum air cavity temperatures).

Table 6.2 gives a summary of the amount that the maximum daily brick surface temperature exceeds the ambient temperature. The table also indicates the incremental temperatures above which a certain percentage of occurrences lie. For example, on 50% of the days observed, the brick surface temperature in the south wall rose at least 9°C above the ambient air temperature.

Table 6.2 Summary of brick surface and air cavity incremental temperatures.

Wall	Amount that Brick Surface Temp. Rises Above Ambient ($^{\circ}\text{C}$)			Amount that Air Cavity Temp Rises Above Ambient ($^{\circ}\text{C}$)		
	Maximum	10 Percentile	50 Percentile	Maximum	10 Percentile	50 Percentile
East	31	25	10	15	9	4
South	31	20	9	13	8	3
West	27	15	5	15	8	4
North	13	5	2	9	4	2

Figure 6.4 shows the typical variations occurring in the temperatures of the brick surface, air cavity, and ambient air over a 5 day period. Note that the maximum air cavity temperature generally lags the maximum brick surface temperature by about 2 hours.

Figure 6.4 Temperature variation in the south wall over a five day period.

Figure 6.5 gives the probability of the brick surface temperature rising by a certain amount above the ambient air temperature on any day of the year. For instance, the probability that the east wall brick surface temperature will exceed the ambient by 30°C is about 2%. In other words, a 30°C temperature differential will likely occur during about 7 days out of the year. Figure 6.6 represents the corresponding probability distribution for the air cavity-ambient air temperature differential.

In Figure 6.7 the probability that the maximum brick surface temperature will occur at a certain time of day is presented. It can be seen that the east wall maximum brick surface temperature will most likely occur at 12:00 noon, at about 3:00 PM for the south wall, 6:00 PM for the west wall, and about 4:00 PM for the north wall.

Figure 6.5 Amount that the maximum daily brick surface temperature exceeds the ambient air temperature versus probability of occurrence.

Figure 6.6 Amount that the air cavity temperature exceeds the ambient temperature each day versus probability of occurrence.

Figure 6.7 Time of day at which the brick surface temperature reaches its maximum versus probability of occurrence

6.2.1 Discussion of Temperature Observations

The measured ambient temperatures taken during the course of the year agree well with the annual record published for the Ottawa area by the weather bureau. During 1982, the weather bureau recorded air temperatures ranging from 34°C to -31°C . A slightly smaller variation of 33°C to -29°C was observed at the building site over the year, which may be due to peaks being missed during the two hour sampling period.

A review of Table 6.1 indicates that the east and west walls attain the highest surface temperatures during the summer. This is because the angle of incidence of the solar radiation is close to the wall normal at sunrise for the east wall and sunset for the west wall. The highest thru-wall average temperature of 49°C was observed in the west wall brick veneer. On an annual basis, the average brick veneer temperature varied from minimum to maximum by 56°C in the north wall to 72°C in the west wall.

In Table 6.2, a means of estimating the temperature rise in the veneer due to solar radiation is presented. It must be remembered however that these values are only applicable to red-brown clay brick veneers. Smaller values would likely be observed in lighter coloured walls. Since these incremental temperatures are observed on a daily basis, it is possible to estimate the frequency and severity of daily thermal movement. This in turn allows the designer to estimate the fatigue life of the metal ties connecting the veneer to the backup. For example, if weather bureau data for the location considered indicates that the ambient temperature fluctuates by 10°C on each of at least 50% of the days of the year, and the average veneer temperature rises by a further 7°C (as seen for the east wall in Table 6.2), then the veneer experiences a 17°C temperature differential every second day (at least). This corresponds to approximately 3 mm of differential movement every second day at the roof level of the instrumented structure, assuming that the blockwork is fully restrained by a return wall (the daily temperature fluctuation in the blockwork is relatively small). According to Bergquist (9), a 4 mm diameter tie spanning a 25 mm air cavity subjected to a 3 mm differential movement every second day would last less than 5 years. If the less frequent, but more severe thermal conditions are also included, the fatigue life of the tie will be further decreased. Note that Bergquist recommends that the tie deflection due to temperature movements be limited to less than 0.3 mm in this example (increasing to 5 mm if a 100 mm air cavity is provided).

The two hour lag between the brick surface and air cavity daily peak temperatures shown in Figure 6.4 illustrate the effect of 'thermal mass' on the rate of heat flow through the wall section. In general the amplitude of the air cavity temperature is less than that of the brick surface and, due to the heat storage capacity of the wall, the temperatures of both the brick surface and the air cavity tend to stay above the ambient temperature.

Figures 6.5, 6.6, and 6.7 show the probability distributions of brick surface and air cavity incremental temperatures and their times of occurrence, respectively, based on approximately 250 days of observations. Although Table 6.2 summarizes the data presented in these figures, it should be noted that the 'percentile' values are based on a cumulative probability of occurrence.

6.2.2 Limitations of Measurements

The results of this study are probably applicable to most clay brick veneers, however there are several factors which should be considered for different situations,

- 1) the colour of the brick will have a significant effect on incremental temperatures due to solar radiation,
- 2) the building location and wall orientation will affect not only the angle of incidence of the solar radiation but also its intensity, duration, and frequency,
- 3) shading from balconies, projecting return walls, and other buildings must be considered on an individual basis,
- 4) the reflectance of the foreground will affect the magnitude of the radiant energy striking the wall,
- 5) insulation location in the wall section has a relatively small influence on the brick veneer temperature (unless placed on the outside) but a large influence on blockwork temperature and thermal movements

6.3 Predicted Vertical Differential Movement at Roof Level

In order to gain some appreciation for the magnitude of the differential movement between the veneer and the blockwork frame occurring over time, the loadbearing masonry structure described in the previous section was modelled using the program developed for this report. Differential movement at the roof level will arise as a result of the cumulative effects of creep, shrinkage, elastic, moisture, freezing, and thermal movements occurring at all levels in both the veneer and the frame. These effects can be considered cumulative because the veneer runs uninterrupted from the ground level to the roof and the ties provide negligible resistance to differential movement.

Since thermal movements not only potentially make up a large portion of the total differential movement but also contribute to tie fatigue (due to its cyclic nature), it is of interest to predict how thermal movements are affected by the location and efficiency of the insulation. For this reason, trial runs were performed for three different insulation thicknesses (25, 50, and 75 mm) placed either on the interior face of the blockwork or in the air cavity, as shown in Figure 6.8

Figure 6.8 Model wall cross-sections with;
 (a) insulation placed on the interior face of the blockwork, and
 (b) insulation placed in the air cavity.

The orientation of the trial wall was assumed to be due south, located in Ottawa, Canada, and 12 storeys in height. The storey height used was 2.95 m and the wall was assumed to be partially shaded by adjacent balconies over 1/3 of its height. A plan view of the wall and its tributary area is shown in Figure 6.9 along with the assumed dead and live loads.

Figure 6.9 Plan view of exterior wall section.

Construction of the building began on May 1, 1973 and was completed by Nov. 27, 1973. Differential movement at the roof level was predicted for May 1, 1974, July 1, 1976, July 1, 1978, Jan. 1, 1980, and July 1, 1982 or at approximately 1, 3, 5, 7, and 9 years from the beginning of construction. Table 6.3 lists the pertinent design data used in the model.

Table 6.3 Design data used in the loadbearing masonry structure test case.

Parameter	Assumed Value	Comment
Reflectance of foreground	0.20	grass, swimming pool, houses on a hot summer day building is not air conditioned
Average windspeed	0.5 m/sec	
Interior temp. during winter	20 °C	
Average annual temperature	6 °C	
Annual temperature range	32 °C	not particularly severe
Minimum winter temperature	-30 °C	
Summer dawn temperature	18 °C	
Summer afternoon temperature	28 °C	
Average annual relative humidity	75%	
Brick unit kiln-to-wall time lag	30 days	
Block unit factory-to-wall time lag	60 days	
Time to place each level	14 days	
Lag between blockwork placement and upper slab placement	7 days	
Lag between blockwork placement and brick veneer placement	28 days	
Ratio of modular to nominal vertical brick dimensions	0.851	during the summer during the winter
Density of clay brick	2300 kg/m ³	
Emissivity of brick	0.90	
Absorptivity of brick	0.80	
Specific heat of brick	0.90 kJ/kg · °C	
Thermal conductivity of brick	2.04 W/m · °C 1.74 W/m · °C	
Coefficient of thermal expansion of veneer	5.2-μm/m · °C	
Brick unit moisture expansion	0.20 mm/m	
Total veneer shrinkage at 20 years due to mortar shrinkage	-0.06 mm/m	
Permanent freezing expansion	0.10 mm/m	
Brick unit compressive strength	70 MPa	in the vertical direction at 500 days from creation
Elastic modulus of veneer	18000 MPa	

Table 6.3 (cont'd) Design data used in the loadbearing masonry structure test case.

Parameter	Assumed Value	Comment
Ratio of modular to nominal vertical block dimensions	0.95	
Density of concrete block	2000 kg/m ³	
Specific heat of blockwork	1.03 kJ/kg · °C	
Thermal conductivity of blockwork	1.32 W/m · °C 1.15 W/m · °C	during the summer during the winter
Coefficient of thermal expansion of blockwork	9.2 µm/m · °C	in the vertical dir.
Ultimate blockwork shrinkage	-0.180 mm/m -0.017 mm/m -0.028 mm/m -0.225 mm/m	due to block shrinkage due to mortar shrinkage due to grout shrinkage total
Blockwork reinforcement	15M at 1200mm	
Coefficient of thermal exp. of steel reinforcement	12.06 µm/m · °C	
Block unit compressive strength (based on net area)	19.7 MPa 13.4 MPa	from ground to 3rd floor from 4th floor to roof
Grout compressive strength	11.0 MPa	
Elastic modulus of blockwork	11400 MPa 9740 MPa	from ground to 3rd floor from 4th floor to roof
Insulation type	styrofoam	
Insulation thermal conductivity	0.043 W/m · °C	at 20 °C
Wall finish type	Gypsum board	12 mm thick
Wall finish thermal cond.	0.14 W/m · °C	

A summary of the predicted differential movements at the roof level at the various building ages for various insulation locations and thicknesses is presented in Table 6.4. Also included are the predicted annual and daily differential movements due to temperature variations only. Figure 6.10 shows the summer differential movement results presented in Table 6.4 in a graphical form for 25 mm insulation thickness only (the summer differential movement is not significantly effected by insulation thickness). Figure 6.11 shows the corresponding winter differential movements predicted for insulation thicknesses of 25 and 75 mm. These curves are approximate since they are based on only two points. In Figure 6.12, the daily and annual thermal differential movements expected at the roof level are presented in a graphical form.

Table 6.4 Differential movement at roof level between brick veneer and blockwork frame in mm.

Insulation Location	Interior Face of Blockwork			Air Cavity		
Insulation Thickness	25 mm	50 mm	75 mm	25 mm	50 mm	75 mm
Building Age (yrs) ^a						
1	6.60 ^b	6.79	6.86	7.29	7.51	7.58
3	13.45	13.73	13.79	14.13	14.44	14.51
5	15.97	16.27	16.34	16.65	16.98	17.05
7 (winter)	16.90	18.52	19.28	9.27	8.13	7.43
9	18.50	18.83	18.89	19.18	19.54	19.61
Daily Thermal Movement	3.92 ^c	3.98	4.00	3.84	4.04	4.14
Annual Thermal Movement	0.33	0.97	1.66	8.58	10.07	10.83

a - measured from beginning of construction (approximate)

b - positive value indicates brick rising above block

c - cyclic differential movement with brick and block moving up and down around their mean positions.

The individual components (thermal, shrinkage, creep, etc.) of the absolute movement occurring in the brick veneer and blockwork at the roof level are shown in Tables 6.5 and 6.6 for insulation thicknesses 25 and 75 mm only. The absolute movement is measured relative to the position of the brick or block at the time of their placement. Since each level of brick or block was placed at different times, the 'absolute movement' will not in fact be actually observed at any level, but is rather the total sum of the movements occurring at each level. For example, by the time the brick veneer is placed at the 12th floor, the veneer at, say, the 4th floor has expanded vertically by x mm. The mason, however, does not know this and will place the 12th floor brick at the correct elevation. Thus the absolute movement in the brick veneer at the 12th floor will be at least x mm (the sum of movements in the lower levels) as defined here, whereas the net differential movement will be zero at this time. Negative signs in Tables 6.5 and 6.6 indicate shortening of the structural element.

Figure 6.10 Predicted roof level summer differential movement with 25 mm of insulation.

Figure 6.11 Predicted roof level winter differential movement with 25 and 75 mm of insulation.

Figure 6.13 Predicted roof level daily and annual thermal differential movements.

Table 6.5 Components of the absolute movement in the brick veneer and the blockwork frame at the roof level (insulation on the interior face of the blockwork).

25 mm of Insulation on Interior Face of Blockwork										
Building Age (yrs)	Brick Veneer Movement (mm)				Differential Movement ^c	Blockwork Frame Movement (mm)				Total ^b
	Thermal	Moisture	Elastic	Freezing		Thermal	Moisture	Creep	Elastic	
1	4.43	1.54	-0.07 ^a	3.54	6.60	1.65	-3.00	-3.24	-3.93	-8.52
3	4.43	4.63	-0.08	3.54	13.45	1.65	-5.25	-4.84	-3.84	-12.29
5	4.43	5.97	-0.08	3.54	15.97	1.65	-5.96	-5.35	-3.82	-13.47
7 (winter)	-7.28	6.67	-0.08	3.54	18.90	-9.80	-6.22	-5.58	-3.79	-25.30
9	4.43	7.54	-0.08	3.54	18.50	1.65	-6.46	-5.83	-3.79	-14.44
75 mm of Insulation on Interior Face of Blockwork										
1	4.41	1.54	0	3.54	6.86	1.48	-3.00	-3.37	-3.94	-8.83
3	4.41	4.83	0	3.54	13.79	1.48	-5.25	-5.05	-3.85	-12.67
5	4.41	5.97	0	3.54	16.34	1.48	-5.96	-5.58	-3.82	-13.89
7 (winter)	-7.65	6.67	0	3.54	19.28	-12.35	-6.22	-5.82	-3.80	-28.19
9	4.41	7.54	0	3.54	18.89	1.48	-6.46	-6.08	-3.80	-14.87

a - this trial run uses a small tie stiffness of 8 kN/mm acting at each floor level

b - sum of total movement at each level since time of placement

c - differential movement occurring since placement of roof level brick veneer

Table 6.6 Components of the absolute movement in the brick veneer and the blockwork frame at the roof level (insulation placed in the air cavity).

25 mm of Insulation Placed in Air Cavity										
Building Age (yrs)	Brick Veneer Movement (mm)				Differential Movement ^c	Blockwork Frame Movement (mm)				Total ^b
	Thermal	Moisture	Elastic	Freezing		Thermal	Moisture	Creep	Elastic	
1	4.91	1.54	-0.06 ^a	3.54	7.29	1.44	-3.00	-3.23	-3.94	-8.72
3	4.91	4.83	-0.07	3.54	14.13	1.44	-5.25	-4.82	-3.84	-12.48
5	4.91	5.97	-0.08	3.54	16.65	1.44	-5.96	-5.33	-3.82	-13.67
7 (winter)	-7.27	6.67	-0.08	3.54	9.27	-2.30	-6.22	-5.66	-3.80	-17.78
9	4.91	7.54	-0.08	3.54	19.18	1.44	-6.46	-5.81	-3.79	-14.63
75 mm of Insulation Placed in Air Cavity										
1	5.10	1.54	0	3.54	7.58	1.42	-3.00	-3.35	-3.94	-8.86
3	5.10	4.83	0	3.54	14.51	1.42	-5.25	-5.03	-3.85	-12.71
5	5.10	5.97	0	3.54	17.05	1.42	-5.96	-5.57	-3.82	-13.91
7 (winter)	-7.65	6.67	0	3.54	7.43	-0.51	-6.22	-5.81	-3.80	-16.34
9	5.10	7.54	0	3.54	19.61	1.42	-6.46	-6.07	-3.79	-14.90

a - this trial run uses a small tie stiffness of 8 kN/mm acting at each floor level

b - sum of total movement at each level since time of placement

c - differential movement occurring since placement of roof level brick veneer

6.3.1 Discussion of Predicted Vertical Movement

A review of Table 6.4 and Figure 6.10 indicates that the summer differential movement is not significantly affected by the position and thickness of the insulation. This is also true of the daily differential movement as shown in Table 6.4 and Figure 6.12. The summer ambient temperature tends to oscillate around a mean temperature which is very close to the interior air temperature and, as a result, all elements inside the brick veneer will tend to oscillate around the same temperature but with a reduced amplitude. This means that the blockwork temperature does not change significantly no matter where the insulation is placed (compared to the veneer temperature change). Thus it could be expected that the summer differential movement will not be significantly affected by the insulation location and thickness.

The mean winter temperature, however, is far below the interior air temperature and thus the location of the insulation and, to a lesser degree, the insulation thickness will have a significant effect on the differential movement observed during the winter, as seen in Table 6.4 and Figure 6.11. The winter differential movement is predicted to be between 8 and 12 mm greater (depending on insulation thickness) if the insulation is placed on the interior face of the blockwork than if it is placed in the air cavity. On this basis it appears desirable to place the insulation in the air cavity. The cyclic annual movement, though, is also affected, jumping from about 1 mm for insulation placed on the interior face of the blockwork, to about 10 mm for insulation placed in the air cavity (see Table 6.4 and Figure 6.12). Although placing the insulation in the air cavity results in smaller winter differential displacements it does so at the expense of larger annual cyclic movements. Placing the insulation in the air cavity will be beneficial, however, in reducing the shear forces arising between the external loadbearing blockwork wall and any rigidly connected interior return walls.

It was of interest to find that the annual thermal movement, with insulation placed on the interior face of the blockwork, was such a small value (about 1 mm compared to the daily movement of about 4 mm). If the assumed coefficients of thermal expansion of the veneer and the blockwork, which are mean values derived from the literature, are compared (see Table 6.3), it can be seen that the blockwork value of $9.2 \mu\text{m}/\text{m} \cdot ^\circ\text{C}$ is about 1.8 times the veneer value of $5.2 \mu\text{m}/\text{m} \cdot ^\circ\text{C}$. During the course of a year, the predicted temperature range experienced by the veneer is about 65°C which, coincidentally enough, is 1.85 times the temperature range of 35°C experienced by the blockwork. Thus, although the blockwork goes through a smaller temperature range, it moves just as much as the veneer on an annual basis because of its higher coefficient of thermal expansion. This phenomenon can also be seen in Table 6.5 if the absolute thermal movement occurring in the summer is compared with that occurring in the winter for both the veneer and the blockwork.

In both trial runs (Tables 6.5 and 6.6) with an insulation thickness of 25 mm, an effective tie shear stiffness of about $8 \text{ kN}/\text{mm}$ was applied at each floor level which resulted in a small elastic and creep (included with the elastic value) movement in the veneer of about -0.08 mm at the roof level. A correspondingly small change in the elastic and creep movements of the blockwork can be seen if the 25 mm trial runs are compared to the 75 mm trial runs. Moisture and freezing

expansion of the veneer, as well as shrinkage of the blockwork, are seen to be entirely independent of the insulation location and thickness (as long as the insulation is not on the outer surface), as would be expected. The creep and elastic movements in the blockwork are affected to a small degree by the insulation thickness and position because of the difference in thermal expansion between the concrete block unit and its reinforcement.

The ratio of the blockwork creep strain, at 9 years, to the elastic strain is about 1.6. NCMA (76) gives creep to elastic strain ratios between 1.0 and 1.8 for high strength block, Lenczner (65) gives values between 3.0 and 7.0 for low strength block, and Sutherland (107) gives values between 1.0 and 6.0. Considering the effect of incremental loading (which tends to reduce creep) as well as the restraining effect of the reinforcement, the predicted creep to elastic strain ratio of 1.6 appears quite reasonable.

Tables 6.5 and 6.6 show the elastic movement in the blockwork gradually decreasing by about 4% over the 9 year period considered. This may appear surprising, in that the building loads remain constant, until it is remembered that creep and shrinkage in the blockwork results in a gradual transfer of load to the reinforcement which is about 20 times as stiff as the blockwork ($E_{MBL} \approx 10\,000\text{ MPa}$, and $E_{st} = 200\,000\text{ MPa}$).

In summary it must be pointed out that the results presented in this chapter are only strictly applicable to 12 storey loadbearing masonry apartment buildings with properties similar to those outlined in Table 6.3. However, since the structure considered is quite typical of many loadbearing masonry structures found in Canada, except perhaps in height, the results (suitably scaled for height) serve as a good indication of the differential movement to be expected between the veneer and the frame at the roof level. In general a long term differential movement of about 1.6 mm per storey, an annual differential movement of 0.85 mm per storey (for insulation in the air cavity), and a daily differential movement of 0.33 mm per storey was predicted. Note that the thermal movements are based on a moderate brick veneer coefficient of thermal expansion of $5.2\text{ }\mu\text{m/m}^\circ\text{C}$. In Table 3.10, Grimm (40) gives values as high as $12.4\text{ }\mu\text{m/m}^\circ\text{C}$ which would give an annual differential movement of up to 2.0 mm per storey and a daily differential movement of about 0.8 mm per storey.

Chapter 7

Test Case Two

Reinforced Concrete Structure

7.1 Introduction

In 1979 an eighteen storey apartment building in the Ottawa area was investigated by Seddiek (98). The building is a flat plate and column structure with concrete shear walls to resist lateral loads and is clad with a clay brick veneer tied to a concrete blockwork infill. The brick veneer is supported at each floor level by shelf angles.

The brick veneer was observed to have suffered considerable distress in the lower levels, typified by severe vertical cracks (especially at the corners) and some spalling of the brick surface. At the time the distress was first observed, the building was only about 2 years old.

In this chapter the computer program developed by the author was used to model the differential movements occurring in a typical exterior wall section and derive the resulting axial stresses in each of the structural elements, particularly the brick veneer. Unfortunately a number of structural and environmental parameters were unavailable and had to be assumed. In general the unknown parameters were assigned values which were believed to be typical.

Several parameters were also chosen as variables to evaluate their effect on the axial stress generated in the brick veneer. First the building was modelled under two overall conditions,

- 1) Moderate conditions: all building parameters assume typical values
- 2) Severe conditions: ambient temperature conditions, ultimate moisture strains, and brick veneer radiation absorptivity and freezing expansion parameters assume relatively severe values

Within these overall conditions several other parameters were varied;

- i) age of the building at the time of stress evaluation,
- ii) thickness of the expansion gap provided under each shelf angle,
- iii) stiffness and yield point of the shelf angles, and
- iv) insulation thickness and location.

To include all the possible combinations of the varied parameters would require a very large number of computer runs. In most cases, however, the effect of a given parameter does not change significantly with changes in other parameters (ie., although magnitudes may change, the general

trend does not). Therefore, to simplify the procedure, some parameters were varied for only certain cases. A complete description of the structural and environmental parameters used to model the building considered, as well as the parameters varied, appears in Section 7.2. The computer results are presented and discussed in Section 7.3.

7.2 Structural and Environmental Parameters

As discussed earlier, the building considered is an 18 storey reinforced concrete flat plate and column structure with one basement level. The brick veneer consists of a 92 mm brick wythe tied to a 140 mm concrete blockwork infill panel. Rigid insulation, 25 mm thick, followed by 12 mm of drywall are placed on the interior face of the blockwork and column. A typical cross-section of the exterior wall is shown in Figure 7.1 and a plan view of the panel considered is shown in Figure 7.2. The structural and environmental parameters and their values used in the program are listed in Table 7.1. Figure 7.3 shows a typical shelf angle detail.

Figure 7.1 Cross-section through exterior wall at a non-column (or midbay) location

Figure 7.2 Plan of a typical floor panel.

Figure 7.3 Typical shelf angle connection (from Seddeik, 98).

Table 7.1 Structural and environmental parameters used in the program.

Parameter	Assumed Value	Comment
Storey height	2.64 m	
Tributary area	15.87 m ²	
Dead load on each level	5.51 kN/m ²	
Live load on each level	1.92 kN/m ²	
Fraction of live load;		
a) contributing to creep	0.20	
b) acting when stress is evaluated	0.50	
Orientation of wall considered	South	Azimuth = 180°
Wind exposure condition	Open	Exposure A (NBC)
Interior air temperature	20 °C	air-conditioned all year
Average annual air temp.	8 °C	
Annual ambient temp. range	36 or 42 °C	see Table 7.4
Max. summer afternoon air temp.	30 or 35 °C	see Table 7.4
Corresponding dawn air temp.	29 °C	
Minimum winter air temp.	-20 or -30 °C	see Table 7.4
Average windspeed	1.0 m/sec	constant
Mean annual relative humidity	75%	constant
Window condition	all closed	
Hor. and vert. shading angles	none	
Ground reflectance coefficient	0.20	for all levels
Shelf angle locations	each level	
Shelf angle expansion gap	0, 0.5, 1.0, ... mm	variable parameter
Shelf angle shear stiffness	100 to 500	variable parameter (kN/mm/m) (see Table 7.2)
Shelf angle yield point	30 to 170	variable parameter (kN/m)
Ratio of volume of iron oxide to volume of iron	2.10	
Surface thickness corroded on upper side of shelf angle	0.5 mm	after 3650 days

Table 7.1 (cont'd) Structural and environmental parameters used in the program.

Parameter	Assumed Value	Comment
Brick Veneer;		
a) Tributary width	3.49 m	
b) Thermal conductivity	1.95 W/m · °C	during the summer
	1.51 W/m · °C	during the winter
c) Thickness	92 mm	
d) Specific heat	0.95 kJ/kg · °C	
e) Emissivity	0.90	
f) Solar absorptivity	0.80 to 0.90	see Table 7.4
g) Density	2300 kg/m ³	
h) Coef. of thermal exp.	7.05 µm/m · °C	
i) Ratio of modular to nominal vertical dimensions	0.851	
j) Ult. moisture expansion	0.30 to 0.40 mm/m	see Table 7.4
k) Ult. mortar shrinkage	-0.054 mm/m	contribution to wall shrinkage
l) Brick unit comp. strength	68 MPa	bricks laid 'wet'
m) Elastic modulus of veneer	17892 MPa	at 28 days
n) Freezing expansion strain	0 to 0.10 mm/m	at -7 °C, see Table 7.4
Blockwork;		
a) Tributary width	2.73 m	
b) Thickness (modular)	140 mm	
c) Percent solid	56%	
d) Percent grouted	0%	
e) Thermal conductivity	1.17 W/m · °C	
f) Density	2000 kg/m ³	
g) Specific heat	1.19 kJ/kg · °C	
h) Emissivity	0.90	
i) Absorptivity	0.57	
j) Coef. of thermal exp.	9.2 µm/m · °C	
k) Ult. wall shrinkage due to;		
1) block unit shrinkage	-0.38 to -0.57 mm/m	see Table 7.4
2) mortar joint shrinkage	-0.018 mm/m	
l) Ratio of modular to nominal vertical dimensions	0.95	
m) Block unit comp. strength	17.0 MPa	based on net area
n) Mortar comp. strength	15.0 MPa	
o) Elastic modulus	10375 MPa	at 28 days
p) Reinforcement	none	
q) Gap left above block	0 mm	
Wall Finish;		
a) Wall finish type	gypsum board	
b) Density	960 kg/m ³	
c) Thickness	12.7 mm	
d) Specific heat	1.09 kJ/kg · °C	
e) Thermal conductivity	0.136 W/m · °C	

Table 7.1 (cont'd) Structural and environmental parameters used in the program.

Parameter	Assumed Value	Comment
Insulation; a) Type b) Thickness c) Thermal conductivity	styrofoam 25 to 50 mm 0.043 W/m °C	thickness and location variable at 20 °C
Concrete Column; a) Thermal conductivity b) Density c) Specific heat d) Emissivity e) Absorptivity f) Coef. of thermal exp. g) Ult. concrete shrinkage h) Thickness i) Width j) Compressive strength k) Reinforcement area l) Concrete elastic modulus m) Steel elastic modulus n) Steel coef. of thermal exp.	2.01 W/m °C 1.67 W/m °C 2300 kg/m ³ 0.94 kJ/kg °C 0.90 0.90 9.51 µm/m °C -0.40 to -0.80 mm/m 300 mm 250 mm 750 mm 30 MPa 10 000 mm ² 8000 mm ² 6000 mm ² 5000 mm ² 3900 mm ² 2600 mm ² 2000 mm ² 36900 MPa 200 000 MPa 12.06 µm/m °C	during the summer during the winter see Table 7.4 from basement to 9th floor from 10th floor to roof from basement to roof at 28 days basement ground floor 2nd floor 3rd floor 4th floor 5th to 9th floors 10th floor to roof at 28 days
Date construction started	July 1, 1962	
Date construction completed	Feb. 18, 1963	
Time required to complete each level	10 days	
Level that frame construction had started when block first placed	5	blockwork started in basement
Level that frame construction had started when brick first placed	6	veneer started at ground level
Time lag between veneer and shelf angle placement	0 days	
Kiln-to-wall time lag for brick	30 days	
Factory-to-wall time lag for block	60 days	
Time lag between column placement and shoring removal	30 days	

Stresses in the exterior wall section were evaluated at five different times in the life of the building: 1, 3, 5, and 10 years from the beginning of construction, and again at 10 years from completion of construction. The first four stress calculations take place during the summer (on July 1) and are performed using the specified 'summer day' ambient temperatures and windspeeds (in this case constant at 1.0 m/sec). The last, at 10 years from the building completion date, is performed during the winter (on Feb. 18) and uses the specified minimum winter ambient air temperature.

The expansion gap specified to be provided under each shelf angle was varied from 0.0 to 5.0 mm, in 0.5 to 1.0 mm steps, for two different shelf angle stiffness and yield values as well as for varying insulation thickness and location. In all cases, though, the lowermost brick panel was assumed to be resting directly on the foundation. To determine the effect of shelf angle stiffness parameters on the stresses generated in the building elements, five different shelf angle stiffness-yield point values were run with no expansion gap provided and 25 mm of insulation placed first on the interior face of the blockwork and, second, in the air cavity. The stiffness-yield point values selected are shown in Table 7.2 along with the shelf angle geometry which gives approximately the required stiffness parameters (as derived from Yura, 119).

Table 7.2 Shelf angle stiffness-yield parameters used in the program along with the corresponding angle geometry (see Figure 7.4).

Stiffness (kN/mm/m)	Yield (kN/m)	L (mm)	t (mm)	d (mm)	f_y (MPa)
500	170	120	6.35	9	300
400	150	85	6.35	50	300
300	100	115	5.60	15	250
200	70	90	4.60	40	250
100	30	110	2.30	5	250

The shelf angle geometries shown in Table 7.2 (in conjunction with Figure 7.4) are for demonstrative purposes only. Many combinations of shelf angle thickness (t), leg length (L), and brick overhang (d), all giving the same stiffness-yield values, exist. The geometries given in Table 7.2 have been 'fine tuned', assuming the bolted connection to be infinitely stiff, to give the assumed stiffness-yield values but represent only one possible solution. The stiffness parameters derived for some standard equal leg angles are shown in Table 7.3, based on the following assumptions;

- 1) the bolted connection is infinitely stiff,
- 2) the steel grade is 300 MPa, and
- 3) the brick overhang (d) is 25 mm.

Figure 7.4 Shelf angle geometry used to determine stiffness-yield parameters.

Table 7.3 Stiffness parameters of some standard equal leg angles.

Angle ($L \times L \times t$, mm)	Stiffness ($kN/mm/m$)	Yield (kN/m)
150 × 150 × 10	200	180
125 × 125 × 10	560	260
× 8	290	160
100 × 100 × 10	2670	450
× 8	1370	290
× 6	580	160

All stiffness and yield values have been expressed per metre length of the shelf angle.

The effect of insulation thickness and location on the stresses developed in the exterior wall section was investigated as follows;

- 1) Two different insulation thicknesses (25 and 50 mm) placed in two different locations (air cavity and interior face of blockwork) were run for shelf angle stiffnesses of 300 and 400 kN/mm/m, varying expansion gap, and varying building age.
- 2) 25 mm of insulation placed in two different locations (air cavity and interior face of blockwork) were run for varying shelf angle stiffness and varying building age with an expansion gap of zero.

The effects due to variations in building age, expansion gap thickness, shelf angle stiffness-yield, and insulation thickness and location were evaluated under two overall conditions: moderate and severe, as outlined in Section 7.1. In Table 7.4 the difference between these conditions is presented.

Table 7.4 Parameters varied under moderate and severe conditions.

Parameter	Condition	
	Moderate	Severe
Minimum winter temp.	-20 °C	-30 °C
Summer afternoon temp.	30 °C	35 °C
Annual temp. range	36 °C	42 °C
Ult. concrete shrinkage	-0.40 mm/m	-0.80 mm/m
Brick solar absorptivity	0.80	0.90
Ult. brick moisture exp.	0.30 mm/m	0.40 mm/m
Brick freezing expansion	0.0 mm/m	0.1 mm/m
Ult. block shrinkage	-0.40 mm/m	-0.60 mm/m

7.3 Predicted Axial Stress in Exterior Wall Elements

In this section the results of the computer analysis on the 18 storey reinforced concrete structure test case are presented. In all trial runs it was observed that the axial stress developed in the brick veneer was highest at the ground level, as would be expected, and so only these stresses are shown in this report. However, the variation in brick veneer stress with height will be shown graphically for two different shelf angle stiffness-yield values.

Tables 7.5 to 7.8 show the predicted axial stresses at ground level in each of the elements of the exterior wall, including the column reinforcement, under moderate conditions for;

- i) shelf angle stiffness of 300 kN/mm per metre length of the shelf angle,
- ii) 25 and 50 mm of insulation placed on the interior face of the blockwork and, subsequently, in the air cavity

Within each table, the stresses are listed for varying building ages and varying expansion gap thicknesses. Negative stress values indicate compression.

Tables 7.9 to 7.12 essentially duplicate the conditions used in Tables 7.5 to 7.8 except that an increased shelf angle stiffness of 400 kN/mm per metre length of the shelf angle has been used (along with its corresponding yield strength).

Under severe conditions, the ground level stresses were evaluated for the four different insulation thickness and position parameters using a shelf angle stiffness of 300 kN/mm/m , as shown in Tables 7.13 to 7.16.

The effect of different shelf angle stiffness-yield parameters on the stresses developed at the ground level is investigated in Tables 7.17 to 7.20. Tables 7.17 and 7.18 were derived under moderate conditions and zero expansion gap for two different insulation locations (25 mm thickness). In Tables 7.19 and 7.20, the same parameters were evaluated under severe conditions.

Table 7.5 Ground level stresses with 25 mm of insulation placed on the interior face of the blockwork and column.

Moderate Conditions

Shelf angle stiffness = 300 kN/mm/m

Shelf angle yield = 100 kN/m

Gap (mm)	Season	Time (yrs.)	Veneer (MPa)	Block (MPa)	Concrete (MPa)	Reinforcement (MPa)
0.0	Summer	1	-6.040	-0.112	-1.427	-59.900
	Summer	3	-8.030	0	1.144	-66.703
	Summer	5	-8.830	0	2.143	-67.298
	Summer	10	-9.847	0	3.308	-65.261
	Winter	10	-8.680	0	1.623	-58.204
0.5	Summer	1	-4.565	-0.401	-2.485	-72.311
	Summer	3	-6.858	0	0.240	-81.019
	Summer	5	-7.665	0	1.267	-82.131
	Summer	10	-8.720	0	2.498	-80.605
	Winter	10	-7.557	0	0.819	-73.576
1.0	Summer	1	-2.952	-0.736	-3.682	-84.295
	Summer	3	-5.158	-0.097	-1.172	-96.386
	Summer	5	-6.384	0	0.256	-97.133
	Summer	10	-7.446	0	1.517	-96.181
	Winter	10	-5.920	-0.308	-0.283	-89.646
1.5	Summer	1	-0.811	-1.222	-5.427	-94.836
	Summer	3	-3.457	-0.459	-2.473	-107.769
	Summer	5	-4.467	-0.211	-1.379	-110.620
	Summer	10	-5.714	0	-0.127	-111.061
	Winter	10	-3.074	-0.680	-2.697	-108.441
2.0	Summer	1	0	-1.413	-6.107	-98.152
	Summer	3	-1.346	-0.931	-4.188	-118.543
	Summer	5	-2.517	-0.637	-2.924	-121.883
	Summer	10	-3.825	-0.329	-1.596	-122.818
	Winter	10	-2.560	-0.764	-3.011	-114.727
3.0	Summer	1	0	-1.413	-6.107	-98.152
	Summer	3	0	-1.248	-5.317	-123.982
	Summer	5	0	-1.225	-5.016	-132.764
	Summer	10	0	-1.214	-4.728	-140.868
	Winter	10	0	-1.348	-5.076	-127.853

Table 7.6 Ground level stresses with 50 mm of insulation placed on the interior face of the blockwork and column.

Moderate Conditions

Shelf angle stiffness = 300 kN/mm/m

Shelf angle yield = 100 kN/m

Gap (mm)	Season	Time (yrs.)	Veneer (MPa)	Block (MPa)	Concrete (MPa)	Reinforcement (MPa)
0.0	Summer	1	-5.987	-0.104	-1.499	-60.023
	Summer	3	-7.976	0	1.068	-66.562
	Summer	5	-8.797	0	2.092	-67.134
	Summer	10	-9.812	0	3.254	-65.077
	Winter	10	-9.071	0	1.945	-54.112
0.5	Summer	1	-4.512	-0.393	-2.557	-72.435
	Summer	3	-6.794	0	0.156	-80.941
	Summer	5	-7.620	0	1.204	-82.044
	Summer	10	-8.674	0	2.434	-80.503
	Winter	10	-7.935	0	1.129	-69.574
1.0	Summer	1	-2.899	-0.728	-3.753	-84.426
	Summer	3	-5.093	-0.091	-1.263	-96.351
	Summer	5	-6.340	0	0.195	-97.045
	Summer	10	-7.405	0	1.459	-96.067
	Winter	10	-6.589	0	0.078	-85.635
1.5	Summer	1	-0.789	-1.206	-5.462	-95.171
	Summer	3	-3.397	-0.451	-2.547	-108.078
	Summer	5	-4.436	-0.196	-1.424	-110.928
	Summer	10	-5.819	0	0.057	-110.553
	Winter	10	-4.855	-0.287	-1.245	-99.499
2.0	Summer	1	0	-1.391	-6.123	-98.398
	Summer	3	-1.327	-0.909	-4.223	-118.913
	Summer	5	-2.499	-0.616	-2.958	-122.238
	Summer	10	-3.808	-0.308	-1.629	-123.176
	Winter	10	-2.983	-0.688	-2.698	-111.327
4.0	Summer	1	0	-1.391	-6.123	-98.398
	Summer	3	0	-1.222	-5.336	-124.275
	Summer	5	0	-1.200	-5.035	-133.033
	Summer	10	0	-1.188	-4.748	-141.141
	Winter	10	0	-1.372	-5.120	-126.137

Table 7.7 Ground level stresses with 25 mm of insulation placed in the air cavity.

Moderate Conditions

Shelf angle stiffness = 300 $kN/mm/m$ Shelf angle yield = 100 kN/m

Gap (mm)	Season	Time (yrs.)	Veneer (MPa)	Block (MPa)	Concrete (MPa)	Reinforcement (MPa)
0.0	Summer	1	-6.918	0	-0.587	-57.145
	Summer	3	-8.681	0	1.941	-66.768
	Summer	5	-9.335	0	2.774	-67.722
	Summer	10	-10.370	0	3.971	-65.937
	Winter	10	-6.289	0	-0.405	-82.337
0.5	Summer	1	-5.293	-0.214	-1.845	-70.555
	Summer	3	-7.569	0	1.091	-80.550
	Summer	5	-8.227	0	1.944	-81.884
	Summer	10	-9.302	0	3.207	-80.558
	Winter	10	-4.753	-0.718	-1.058	-96.301
1.0	Summer	1	-3.678	-0.557	-3.073	-81.541
	Summer	3	-6.112	0	-0.210	-93.489
	Summer	5	-6.923	0	0.837	-95.053
	Summer	10	-8.008	0	2.128	-94.135
	Winter	10	-3.429	-0.744	-2.144	-109.987
1.5	Summer	1	-1.333	-1.096	-5.025	-91.823
	Summer	3	-4.210	-0.271	-1.846	-104.840
	Summer	5	-5.015	-0.072	-0.956	-107.664
	Summer	10	-6.480	0	0.711	-106.300
	Winter	10	-1.909	-0.845	-3.469	-121.667
2.0	Summer	1	0	-1.409	-6.143	-97.275
	Summer	3	-1.872	-0.814	-3.776	-115.367
	Summer	5	-3.035	-0.517	-2.524	-118.783
	Summer	10	-4.338	-0.212	-1.201	-119.705
	Winter	10	-0.021	-1.268	-4.930	-133.240
3.0	Summer	1	0	-1.409	-6.143	-97.275
	Summer	3	0	-1.256	-5.348	-122.935
	Summer	5	0	-1.227	-5.047	-131.850
	Summer	10	0	-1.216	-4.759	-139.955
	Winter	10	0	-1.252	-4.859	-136.309

Table 7.8 Ground level stresses with 50 mm of insulation placed in the air cavity.

Moderate Conditions

Shelf angle stiffness = 300 kN/mm/m

Shelf angle yield = 100 kN/m

Gap (mm)	Season	Time (yrs)	Veneer (MPa)	Block (MPa)	Concrete (MPa)	Reinforcement (MPa)
0.0	Summer	1	-7.020	0	-0.454	-56.934
	Summer	3	-8.786	0	2.079	-67.147
	Summer	5	-9.409	0	2.878	-68.215
	Summer	10	-10.450	0	4.086	-66.498
	Winter	10	-5.854	0	-0.779	-86.745
1.0	Summer	1	-3.790	-0.509	-3.000	-81.205
	Summer	3	-6.220	0	-0.088	-93.335
	Summer	5	-6.958	0	0.874	-95.023
	Summer	10	-8.067	0	2.197	-94.155
	Winter	10	-2.939	-1.023	-2.366	-112.833
2.0	Summer	1	0	-1.388	-6.168	-97.278
	Summer	3	-1.958	-0.773	-3.730	-115.008
	Summer	5	-3.118	-0.476	-2.480	-118.439
	Summer	10	-4.420	-0.170	-1.158	-119.359
	Winter	10	0	-1.255	-4.924	-134.567
4.0	Summer	1	0	-1.388	-6.168	-97.278
	Summer	3	0	-1.235	-5.373	-122.924
	Summer	5	0	-1.205	-5.073	-131.864
	Summer	10	0	-1.194	-4.785	-139.973
	Winter	10	0	-1.234	-4.833	-137.583

Table 7.9 Ground level stresses with 25 mm of insulation placed on the interior face of the blockwork and column.

Moderate Conditions

Shelf angle stiffness = 400 kN/mm/m

Shelf angle yield = 150 kN/m

Gap (mm)	Season	Time (yrs.)	Veneer (MPa)	Block (MPa)	Concrete (MPa)	Reinforcement (MPa)
0.0	Summer	1	-6.026	-0.117	-1.436	-59.976
	Summer	3	-8.021	0	1.136	-66.819
	Summer	5	-8.822	0	2.136	-67.427
	Summer	10	-9.838	0	3.303	-65.402
	Winter	10	-8.673	0	1.619	-58.338
0.5	Summer	1	-4.561	-0.402	-2.488	-72.315
	Summer	3	-6.848	0	0.230	-81.072
	Summer	5	-7.655	0	1.257	-82.190
	Summer	10	-8.710	0	2.489	-80.672
	Winter	10	-7.547	0	0.809	-73.642
1.0	Summer	1	-2.948	-0.737	-3.685	-84.315
	Summer	3	-5.145	-0.100	-1.183	-96.465
	Summer	5	-6.370	0	0.243	-97.233
	Summer	10	-7.433	0	1.506	-96.289
	Winter	10	-5.906	-0.312	-0.292	-89.746
1.5	Summer	1	-0.809	-1.223	-5.429	-94.843
	Summer	3	-3.445	-0.461	-2.483	-107.835
	Summer	5	-4.455	-0.214	-1.389	-110.694
	Summer	10	-5.701	0	-0.137	-111.145
	Winter	10	-4.432	-0.359	-1.556	-103.016
2.0	Summer	1	0	-1.413	-6.107	-98.152
	Summer	3	-1.335	-0.933	-4.197	-118.588
	Summer	5	-2.505	-0.640	-2.934	-121.947
	Summer	10	-3.809	-0.333	-1.609	-122.906
	Winter	10	-2.547	-0.767	-3.021	-114.805
3.0	Summer	1	0	-1.413	-6.107	-98.152
	Summer	3	0	-1.248	-5.317	-123.982
	Summer	5	0	-1.225	-5.016	-132.764
	Summer	10	0	-1.214	-4.728	-140.868
	Winter	10	0	-1.348	-5.076	-127.853

Table 7.10 Ground level stresses with 50 mm of insulation placed on the interior face of the blockwork and column.

Moderate Conditions

Shelf angle stiffness = 400 kN/mm/m

Shelf angle yield = 150 kN/m

Gap (mm)	Season	Time (yrs)	Veneer (MPa)	Block (MPa)	Concrete (MPa)	Reinforcement (MPa)
0.0	Summer	1	-5.973	-0.109	-1.508	-60.100
	Summer	3	-7.967	0	1.061	-66.671
	Summer	5	-8.789	0	2.086	-67.257
	Summer	10	-9.804	0	3.250	-65.212
	Winter	10	-9.064	0	1.942	-54.242
0.5	Summer	1	-4.509	-0.394	-2.560	-72.438
	Summer	3	-6.784	0	0.145	-80.993
	Summer	5	-7.610	0	1.195	-82.102
	Summer	10	-8.664	0	2.424	-80.568
	Winter	10	-7.927	0	1.121	-69.630
1.0	Summer	1	-2.895	-0.729	-3.757	-84.443
	Summer	3	-5.080	-0.094	-1.273	-96.421
	Summer	5	-6.327	0	0.182	-97.134
	Summer	10	-7.389	0	1.443	-96.178
	Winter	10	-6.665	0	0.159	-85.296
1.5	Summer	1	-0.787	-1.206	-5.463	-95.178
	Summer	3	-3.385	-0.453	-2.556	-108.139
	Summer	5	-4.424	-0.199	-1.434	-111.000
	Summer	10	-5.817	0	0.055	-110.596
	Winter	10	-4.843	-0.289	-1.254	-99.575
2.0	Summer	1	0	-1.391	-6.123	-98.398
	Summer	3	-1.315	-0.912	-4.232	-118.958
	Summer	5	-2.487	-0.619	-2.968	-122.301
	Summer	10	-3.792	-0.311	-1.642	-123.265
	Winter	10	-2.971	-0.690	-2.708	-111.402
3.0	Summer	1	0	-1.391	-6.123	-98.398
	Summer	3	0	-1.222	-5.336	-124.275
	Summer	5	0	-1.200	-5.035	-133.033
	Summer	10	0	-1.188	-4.748	-141.141
	Winter	10	0	-1.372	-5.120	-126.137

Table 7.11 Ground level stresses with 25 mm of insulation placed in the air cavity.

Moderate Conditions

Shelf angle stiffness = 400 kN/mm/m

Shelf angle yield = 150 kN/m

Gap (mm)	Season	Time (yrs.)	Veneer (MPa)	Block (MPa)	Concrete (MPa)	Reinforcement (MPa)
0.0	Summer	1	-6.907	0	-0.576	-57.233
	Summer	3	-8.671	0	1.934	-66.904
	Summer	5	-9.321	0	2.764	-67.890
	Summer	10	-10.359	0	3.964	-66.106
	Winter	10	-6.277	0	-0.414	-82.514
0.5	Summer	1	-5.289	-0.215	-1.849	-70.571
	Summer	3	-7.559	0	1.082	-80.623
	Summer	5	-8.214	0	1.932	-81.983
	Summer	10	-9.289	0	3.195	-80.663
	Winter	10	-4.731	-0.723	-1.075	-96.430
1.0	Summer	1	-3.677	-0.557	-3.073	-81.550
	Summer	3	-6.101	0	-0.221	-93.569
	Summer	5	-6.857	0	0.765	-95.274
	Summer	10	-7.966	0	2.087	-94.425
	Winter	10	-3.381	-0.950	-2.020	-109.431
1.5	Summer	1	-1.335	-1.095	-5.024	-91.818
	Summer	3	-4.199	-0.274	-1.856	-104.897
	Summer	5	-5.002	-0.075	-0.966	-107.738
	Summer	10	-6.467	0	0.699	-106.390
	Winter	10	-1.894	-0.849	-3.480	-121.755
2.0	Summer	1	0	-1.409	-6.143	-97.275
	Summer	3	-1.861	-0.817	-3.786	-115.412
	Summer	5	-3.023	-0.520	-2.533	-118.844
	Summer	10	-4.323	-0.215	-1.212	-119.788
	Winter	10	0	-1.273	-4.948	-133.352
2.5	Summer	1	0	-1.409	-6.143	-97.275
	Summer	3	0	-1.256	-5.348	-122.935
	Summer	5	-0.615	-1.082	-4.531	-129.372
	Summer	10	-2.225	-0.691	-2.891	-131.014
	Winter	10	0	-1.253	-4.864	-136.132
3.0	Summer	1	0	-1.409	-6.143	-97.275
	Summer	3	0	-1.256	-5.348	-122.935
	Summer	5	0	-1.227	-5.047	-131.850
	Summer	10	0	-1.216	-4.759	-139.955
	Winter	10	0	-1.252	-4.859	-136.309

Table 7.12 Ground level stresses with 50 mm of insulation placed in the air cavity.

Moderate Conditions

Shelf angle stiffness = 400 kN/mm/m

Shelf angle yield = 150 kN/m

Gap (mm)	Season	Time (yrs.)	Veneer (MPa)	Block (MPa)	Concrete (MPa)	Reinforcement (MPa)
0.0	Summer	1	-7.010	0	-0.463	-57.021
	Summer	3	-8.773	0	2.068	-67.302
	Summer	5	-9.397	0	2.870	-68.375
	Summer	10	-10.439	0	4.079	-66.672
	Winter	10	-5.838	0	-0.790	-86.940
0.5	Summer	1	-5.415	-0.161	-1.757	-70.427
	Summer	3	-7.674	0	1.226	-80.870
	Summer	5	-8.275	0	2.019	-82.435
	Summer	10	-9.360	0	3.295	-81.142
	Winter	10	-4.254	-0.828	-1.409	-100.490
1.0	Summer	1	-3.792	-0.509	-2.998	-81.208
	Summer	3	-6.207	0	-0.100	-93.430
	Summer	5	-6.944	0	0.861	-95.134
	Summer	10	-8.054	0	2.185	-94.284
	Winter	10	-2.923	-1.027	-2.378	-112.953
1.5	Summer	1	-1.420	-1.054	-4.978	-91.471
	Summer	3	-4.323	-0.222	-1.772	-104.559
	Summer	5	-5.085	-0.033	-0.923	-107.387
	Summer	10	-6.539	0	0.765	-105.920
	Winter	10	-1.424	-0.942	-3.852	-124.877
2.0	Summer	1	0	-1.388	-6.168	-97.278
	Summer	3	-1.947	-0.775	-3.739	-115.053
	Summer	5	-3.107	-0.478	-2.490	-118.498
	Summer	10	-4.406	-0.174	-1.169	-119.442
	Winter	10	0	-1.255	-4.923	-134.592
3.0	Summer	1	0	-1.388	-6.168	-97.278
	Summer	3	0	-1.235	-5.373	-122.924
	Summer	5	0	-1.205	-5.073	-131.864
	Summer	10	0	-1.194	-4.785	-139.973
	Winter	10	0	-1.234	-4.833	-137.583

Table 7.13 Ground level stresses with 25 mm of insulation placed on the interior face of the blockwork and column.

Severe Conditions

Shelf angle stiffness = 300 kN/mm/m

Shelf angle yield = 100 kN/m

Gap (mm)	Season	Time (yrs.)	Veneer (MPa)	Block (MPa)	Concrete (MPa)	Reinforcement (MPa)
0.0	Summer	1	-8.549	0	1.832	-68.191
	Summer	3	-10.862	0	5.138	-81.287
	Summer	5	-11.899	0	6.477	-83.262
	Summer	10	-13.126	0	7.910	-81.521
	Winter	10	-11.708	0	5.850	-72.611
0.5	Summer	1	-6.886	-0.040	0.359	-82.229
	Summer	3	-9.699	0	4.219	-94.878
	Summer	5	-10.778	0	5.625	-97.265
	Summer	10	-12.042	0	7.118	-95.929
	Winter	10	-10.626	0	5.062	-87.044
1.0	Summer	1	-5.305	-0.366	-0.806	-94.292
	Summer	3	-8.429	0	3.206	-109.443
	Summer	5	-9.511	0	4.635	-112.398
	Summer	10	-10.816	0	6.202	-111.658
	Winter	10	-9.403	0	4.151	-102.816
1.5	Summer	1	-3.597	-0.731	-2.120	-105.396
	Summer	3	-6.580	0	1.485	-124.440
	Summer	5	-8.152	0	3.489	-126.260
	Summer	10	-9.444	0	5.054	-125.960
	Winter	10	-8.047	0	3.025	-117.186
2.0	Summer	1	-1.471	-1.212	-3.858	-115.829
	Summer	3	-4.874	-0.343	0.170	-135.621
	Summer	5	-6.170	-0.051	1.641	-140.426
	Summer	10	-7.979	0	3.759	-139.342
	Winter	10	-6.136	-0.241	1.460	-131.561
3.0	Summer	1	0	-1.558	-5.091	-121.843
	Summer	3	-0.547	-1.317	-3.364	-157.003
	Summer	5	-2.222	-0.917	-1.500	-162.796
	Summer	10	-3.681	-0.525	0.226	-165.155
	Winter	10	-2.343	-1.061	-1.497	-155.011
4.0	Summer	1	0	-1.558	-5.091	-121.843
	Summer	3	0	-1.446	-3.823	-159.213
	Summer	5	0	-1.439	-3.362	-171.898
	Summer	10	0	-1.424	-2.972	-182.902
	Winter	10	0	-1.595	-3.398	-166.737

Table 7.14 Ground level stresses with 50 mm of insulation placed on the interior face of the blockwork and column.

Severe Conditions

Shelf angle stiffness = 300 kN/mm/m

Shelf angle yield = 100 kN/m

Gap (mm)	Season	Time (yrs.)	Veneer (MPa)	Block (MPa)	Concrete (MPa)	Reinforcement (MPa)
0.0	Summer	1	-8.508	0	1.777	-68.163
	Summer	3	-10.801	0	5.051	-81.057
	Summer	5	-11.867	0	6.424	-82.992
	Summer	10	-13.092	0	7.854	-81.224
	Winter	10	-12.203	0	6.260	-67.451
0.5	Summer	1	-6.833	-0.031	0.285	-82.354
	Summer	3	-9.644	0	4.142	-94.711
	Summer	5	-10.749	0	5.578	-97.074
	Summer	10	-12.013	0	7.071	-95.714
	Winter	10	-11.123	0	5.479	-81.984
1.0	Summer	1	-5.250	-0.357	-0.881	-94.423
	Summer	3	-8.363	0	3.118	-109.356
	Summer	5	-9.466	0	4.573	-112.311
	Summer	10	-10.773	0	6.141	-111.550
	Winter	10	-9.889	0	4.557	-97.845
1.5	Summer	1	-3.543	-0.722	-2.195	-105.536
	Summer	3	-6.514	0	1.391	-124.402
	Summer	5	-8.111	0	3.430	-126.167
	Summer	10	-9.402	0	4.995	-125.856
	Winter	10	-8.534	0	3.433	-112.217
2.0	Summer	1	-1.447	-0.881	-3.895	-116.187
	Summer	3	-4.810	-0.336	0.092	-135.949
	Summer	5	-6.140	-0.036	1.598	-140.753
	Summer	10	-7.949	0	3.729	-139.639
	Winter	10	-6.658	-0.153	1.847	-127.228
3.0	Summer	1	0	-1.534	-5.108	-122.105
	Summer	3	-0.524	-1.294	-3.403	-157.404
	Summer	5	-2.209	-0.893	-1.531	-163.168
	Summer	10	-3.867	-0.500	0.195	-165.548
	Winter	10	-2.880	-0.964	-1.099	-150.722
4.0	Summer	1	0	-1.534	-5.108	-122.105
	Summer	3	0	-1.418	-3.843	-159.524
	Summer	5	0	-1.412	-3.383	-172.181
	Summer	10	0	-1.396	-2.992	-183.188
	Winter	10	0	-1.626	-3.453	-164.555

Table 7.15 Ground level stresses with 25 mm of insulation placed in the air cavity.

Severe Conditions

Shelf angle stiffness = 300 kN/mm/m

Shelf angle yield = 100 kN/m

Gap (mm)	Season	Time (yrs.)	Veneer (MPa)	Block (MPa)	Concrete (MPa)	Reinforcement (MPa)
0.0	Summer	1	-9.312	0	2.663	-65.474
	Summer	3	-11.604	0	6.031	-80.952
	Summer	5	-12.462	0	7.167	-83.331
	Summer	10	-13.706	0	8.629	-81.829
	Winter	10	-8.636	0	3.195	-102.280
0.5	Summer	1	-7.949	0	1.470	-78.244
	Summer	3	-10.456	0	5.122	-95.307
	Summer	5	-11.354	0	6.319	-97.025
	Summer	10	-12.633	0	7.838	-95.900
	Winter	10	-7.566	0	2.409	-116.374
1.0	Summer	1	-6.167	-0.144	-0.059	-91.942
	Summer	3	-9.261	0	4.178	-108.259
	Summer	5	-10.124	0	5.351	-111.486
	Summer	10	-11.456	0	6.953	-110.845
	Winter	10	-6.138	0	1.264	-132.620
1.5	Summer	1	-4.461	-0.518	-1.403	-101.939
	Summer	3	-7.699	0	2.704	-119.998
	Summer	5	-8.670	0	4.020	-123.540
	Summer	10	-10.017	0	5.656	-123.285
	Winter	10	-4.338	-1.260	0.640	-141.642
2.0	Summer	1	-2.132	-1.050	-3.348	-112.098
	Summer	3	-5.773	-0.103	0.905	-132.217
	Summer	5	-7.067	0	2.506	-135.552
	Summer	10	-8.549	0	4.304	-135.248
	Winter	10	-2.867	-1.203	-0.760	-153.866
3.0	Summer	1	0	-1.551	-5.135	-120.815
	Summer	3	-1.201	-1.172	-2.851	-153.136
	Summer	5	-2.850	-0.773	-1.016	-158.973
	Summer	10	-4.493	-0.387	0.693	-161.270
	Winter	10	0	-1.495	-3.203	-174.709
4.0	Summer	1	0	-1.551	-5.135	-120.815
	Summer	3	0	-1.455	-3.859	-157.989
	Summer	5	0	-1.442	-3.398	-170.831
	Summer	10	0	-1.428	-3.006	-181.839
	Winter	10	0	-1.475	-3.124	-177.390

Table 7.16 Ground level stresses with 50 mm of insulation placed in the air cavity.**Severe Conditions**

Shelf angle stiffness = 300 kN/mm/m

Shelf angle yield = 100 kN/m

Gap (mm)	Season	Time (yrs.)	Veneer (MPa)	Block (MPa)	Concrete (MPa)	Reinforcement (MPa)
0.0	Summer	1	-9.433	0	2.795	-65.155
	Summer	3	-11.721	0	6.183	-81.301
	Summer	5	-12.546	0	7.282	-83.787
	Summer	10	-13.795	0	8.754	-82.363
	Winter	10	-8.088	0	2.716	-107.558
1.0	Summer	1	-6.319	-0.083	0.053	-91.668
	Summer	3	-9.389	0	4.334	-108.371
	Summer	5	-10.219	0	5.469	-111.679
	Summer	10	-11.551	0	7.074	-111.109
	Winter	10	-5.239	-0.876	1.140	-135.715
2.0	Summer	1	-2.237	-1.002	-3.287	-111.645
	Summer	3	-5.918	-0.047	1.006	-131.761
	Summer	5	-7.185	0	2.618	-134.786
	Summer	10	-8.634	0	4.380	-134.627
	Winter	10	-2.277	-1.322	-1.226	-157.682
4.0	Summer	1	0	-1.528	-5.163	-120.795
	Summer	3	0	-1.434	-3.885	-157.946
	Summer	5	0	-1.421	-3.425	-170.815
	Summer	10	0	-1.407	-3.032	-181.825
	Winter	10	0	-1.455	-3.091	-178.940

Table 7.17 Ground level stresses with 25 mm of insulation placed on the interior face of the blockwork and column.

Moderate Conditions

Expansion gap = 0.0 mm

Stiffness (kN/mm/m)	Season	Time (yrs.)	Veneer (MPa)	Block (MPa)	Concrete (MPa)	Reinforcement (MPa)
100 (30)	Summer	1	-8.049	-0.106	-1.418	-59.971
	Summer	3	-7.762	0	0.865	-68.007
	Summer	5	-8.013	0	1.295	-71.375
	Summer	10	-8.126	0	1.539	-74.239
	Winter	10	-7.997	0	0.932	-62.054
200 (70)	Summer	1	-8.045	-0.110	-1.423	-59.874
	Summer	3	-8.049	0	1.162	-66.558
	Summer	5	-8.852	0	2.163	-67.139
	Summer	10	-9.868	0	3.328	-65.080
	Winter	10	-8.696	0	1.637	-58.047
300 (100)	Summer	1	-8.040	-0.112	-1.427	-59.900
	Summer	3	-8.030	0	1.144	-66.703
	Summer	5	-8.830	0	2.143	-67.298
	Summer	10	-9.847	0	3.308	-65.261
	Winter	10	-8.680	0	1.623	-58.204
400 (150)	Summer	1	-8.026	-0.117	-1.436	-59.976
	Summer	3	-8.021	0	1.136	-66.819
	Summer	5	-8.822	0	2.136	-67.427
	Summer	10	-9.838	0	3.303	-65.402
	Winter	10	-8.673	0	1.619	-58.338
500 (170)	Summer	1	-8.017	-0.121	-1.442	-60.012
	Summer	3	-8.020	0	1.136	-66.847
	Summer	5	-8.819	0	2.135	-67.474
	Summer	10	-9.836	0	3.302	-65.458
	Winter	10	-8.670	0	1.619	-58.392

Bracketed values are the corresponding yield strengths of the shelf angles in kN/m.

Table 7.18 Ground level stresses with 25 mm of insulation placed in the air cavity.

Moderate Conditions

Expansion gap = 0.0 mm

Stiffness (kN/mm/m)	Season	Time (yrs)	Veneer (MPa)	Block (MPa)	Concrete (MPa)	Reinforcement (MPa)
100 (30)	Summer	1	-6.932	0	-0.549	-57.152
	Summer	3	-7.978	0	1.208	-70.196
	Summer	5	-8.118	0	1.511	-73.768
	Summer	10	-8.127	0	1.663	-77.582
	Winter	10	-6.539	0	-0.125	-81.675
200 (70)	Summer	1	-6.928	0	-0.555	-57.095
	Summer	3	-8.607	0	1.956	-66.626
	Summer	5	-9.353	0	2.790	-67.556
	Summer	10	-10.388	0	3.987	-65.747
	Winter	10	-6.312	0	-0.385	-82.127
300 (100)	Summer	1	-6.918	0	-0.567	-57.145
	Summer	3	-8.681	0	1.941	-66.768
	Summer	5	-9.335	0	2.774	-67.722
	Summer	10	-10.370	0	3.971	-65.937
	Winter	10	-6.289	0	-0.405	-82.337
400 (150)	Summer	1	-6.907	0	-0.576	-57.233
	Summer	3	-8.671	0	1.934	-66.904
	Summer	5	-9.321	0	2.764	-67.890
	Summer	10	-10.359	0	3.964	-66.106
	Winter	10	-6.277	0	-0.414	-82.514
500 (170)	Summer	1	-6.908	0	-0.575	-57.239
	Summer	3	-8.668	0	1.931	-66.957
	Summer	5	-9.318	0	2.761	-67.951
	Summer	10	-10.354	0	3.961	-66.182
	Winter	10	-6.271	0	-0.418	-82.597

Bracketed values are the corresponding yield strengths of the shelf angles in kN/m.

Table 7.19 Ground level stresses with 25 mm of insulation placed on the interior face of the blockwork and column.

Severe Conditions

Expansion gap = 0.0 mm

Stiffness (kN/mm/m)	Season	Time (yrs.)	Veneer (MPa)	Block (MPa)	Concrete (MPa)	Reinforcement (MPa)
100 (30)	Summer	1	-7.984	0	1.232	-70.644
	Summer	3	-8.126	0	2.312	-95.219
	Summer	5	-8.126	0	2.628	-103.784
	Summer	10	-8.127	0	2.880	-110.602
	Winter	10	-8.127	0	2.298	-94.831
200 (70)	Summer	1	-8.560	0	1.846	-68.191
	Summer	3	-10.874	0	5.150	-81.218
	Summer	5	-11.911	0	6.488	-83.181
	Summer	10	-13.130	0	7.913	-81.457
	Winter	10	-11.714	0	5.856	-72.536
300 (100)	Summer	1	-8.549	0	1.832	-68.191
	Summer	3	-10.862	0	5.138	-81.287
	Summer	5	-11.899	0	6.477	-83.262
	Summer	10	-13.126	0	7.910	-81.521
	Winter	10	-11.708	0	5.850	-72.611
400 (150)	Summer	1	-8.520	0	1.801	-68.309
	Summer	3	-10.845	0	5.125	-81.498
	Summer	5	-11.884	0	6.467	-83.499
	Summer	10	-13.113	0	7.904	-81.770
	Winter	10	-11.695	0	5.844	-72.859
500 (170)	Summer	1	-8.519	0	1.799	-68.300
	Summer	3	-10.834	0	5.114	-81.533
	Summer	5	-11.872	0	6.454	-83.549
	Summer	10	-13.102	0	7.892	-81.823
	Winter	10	-11.685	0	5.834	-72.904

Bracketed values are the corresponding yield strengths of the shelf angles in kN/m.

Table 7.20 Ground level stresses with 25 mm of insulation placed in the air cavity.

Severe Conditions

Expansion gap = 0.0 mm

Stiffness (kN/mm/m)	Season	Time (yrs.)	Veneer (MPa)	Block (MPa)	Concrete (MPa)	Reinforcement (MPa)
100 (30)	Summer	1	-8.125	0	1.419	-71.028
	Summer	3	-8.126	0	2.418	-98.093
	Summer	5	-8.126	0	2.746	-106.983
	Summer	10	-8.126	0	3.016	-114.292
	Winter	10	-8.126	0	2.849	-109.769
200 (70)	Summer	1	-9.337	0	2.691	-65.414
	Summer	3	-11.615	0	6.043	-80.919
	Summer	5	-12.471	0	7.176	-83.307
	Summer	10	-13.700	0	8.624	-81.866
	Winter	10	-8.654	0	3.214	-102.205
300 (100)	Summer	1	-9.312	0	2.663	-65.474
	Summer	3	-11.604	0	6.031	-80.952
	Summer	5	-12.462	0	7.167	-83.331
	Summer	10	-13.706	0	8.629	-81.829
	Winter	10	-8.636	0	3.195	-102.280
400 (150)	Summer	1	-9.279	0	2.629	-65.630
	Summer	3	-11.603	0	6.030	-80.965
	Summer	5	-12.462	0	7.167	-83.341
	Summer	10	-13.708	0	8.632	-81.830
	Winter	10	-8.633	0	3.192	-102.302
500 (170)	Summer	1	-9.265	0	2.614	-65.691
	Summer	3	-11.578	0	6.006	-81.118
	Summer	5	-12.437	0	7.142	-83.514
	Summer	10	-13.682	0	8.607	-82.021
	Winter	10	-8.611	0	3.171	-102.479

Bracketed values are the corresponding yield strengths of the shelf angles in kN/m.

7.3.1 Discussion of Computer Results

Blockwork Stresses

The blockwork was assumed unable to resist tensile forces since it had no vertical reinforcement extending into the slab. Thus, as the compressive axial stress in the brick veneer increases, the upper slab tends to rise away from the blockwork infill (which is also shrinking), which in turn tends to lower the axial compressive stress in the blockwork to zero. In general, for smaller expansion gap thicknesses, the blockwork stress reaches zero at an early age, particularly under severe conditions. For larger expansion gaps, in which the veneer remains unloaded, the compressive stress in the blockwork is seen to gradually decrease with time. This is due to a transfer of load to the column reinforcement as the blockwork shrinks and creeps away from the upper slab.

The maximum blockwork compressive stress occurs at an early building age and is seen in Tables 7.5 and 7.13 to be effected only slightly by the different assumptions made under moderate and severe conditions (1.41 to 1.56 MPa at 1 year under each condition respectively).

During the winter the brick veneer generally experiences temperatures considerably lower than either the blockwork or the column, particularly if the insulation is in the air cavity. The resulting shortening of the brick panel tends to lower the upper slab, increasing the compressive stresses in both the blockwork and the column. This phenomenon is also observed in the column even when the brick veneer remains unloaded, although in this case it is due to thermal contraction of the column. Both the column concrete and the reinforcement have larger coefficients of thermal expansion than the blockwork and so tend to contract more in the winter (even though the temperatures of the column and blockwork are approximately equal) resulting in a transfer of load to the blockwork.

The effect of insulation location and thickness on the blockwork stress during the winter can be summarized as follows;

- 1) When the insulation is located in the air cavity, the blockwork compressive stress increase due to thermal contraction of the brick veneer is maximized. However, since the veneer is still picking up some of the load, this does not result in the maximum blockwork stresses (see Tables 7.8 and 7.16).
- 2) When the insulation is located on the interior face of the blockwork and column, the blockwork compressive stress increase due to thermal contraction of the column during the winter is maximized. Since this effect occurs even when the brickwork remains unloaded, it leads to the highest compressive stresses predicted in the blockwork (see Table 7.14).

The average axial stress predicted in the blockwork is in all cases far less than its ultimate compressive strength of about 12 MPa (from Equation (3.82)). However unevenness in the upper course of block or flexibility of the slab (resulting in the load being applied near the column) may

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lead to some local crushing or cracking. Fortunately the blockwork stress is generally predicted to decrease with time and so any distress caused initially will not likely worsen.

Concrete Stress

In this discussion the term 'concrete' is used to imply the column concrete, the reinforcement stresses being considered separately. For expansion gaps sufficiently large so that the brick veneer remains unloaded, the concrete stress is seen to remain compressive, in all cases, but to decrease in magnitude with time. Under moderate conditions (Tables 7.5 to 7.12) the summer compressive stress reduces by 23% from 1 to 10 years of age whereas under severe conditions (Tables 7.13 to 7.16) the stress reduces by 41% over the same interval. This stress reduction is due to the transfer of load from the concrete to the reinforcement as the concrete shrinks and creeps.

For smaller expansion gaps, the brick veneer restrains the downward movement of the upper slab, which, combined with column shortening, tends to put the concrete into tension. As the expansion gap thickness decreases, the concrete goes into tension earlier and the tensile force increases. The largest concrete tensile stress of 8.75 MPa (Table 7.16) was predicted under severe conditions for an expansion gap of zero and 50 mm of insulation placed in the air cavity. The tensile strength of the concrete is not likely to be more than about 3.0 MPa and so the concrete will probably crack. Once a crack occurs, a large portion of the compressive stress in the veneer will be relieved. The assumption that the concrete remains linearly elastic under tension thus appears questionable but the results give a useful indication of the extreme stresses possible. Under moderate conditions, the maximum ground level concrete tensile stress was predicted in Table 7.8 to be 4.09 MPa, or about 53% lower than that predicted under severe conditions.

During the winter, the stress acting on the concrete is seen in all cases to increase in the compressive, or negative, direction from its summer value. There are two reasons for this,

- 1) Due to its higher coefficient of thermal expansion, the reinforcement shortens more than the concrete as the column temperature decreases, resulting in a transfer of load to the concrete. The semi-annual concrete stress change from this source varies from a minimum of 0.04 MPa under moderate conditions with insulation in the air cavity (Tables 7.7 and 7.8) to 0.23 MPa under severe conditions with the insulation on the interior face of the blockwork (Table 7.14).
- 2) When the brick veneer is in contact with the upper shelf angle (for small expansion gaps), its thermal contraction allows the upper slab to lower, resulting in a transfer of load to the column. For an expansion gap thickness of zero, the semi-annual concrete stress change from this source varies from a minimum of 1.31 MPa, under moderate conditions with the insulation placed on the interior face of the blockwork and column (Table 7.6), to a maximum of 6.04 MPa, under severe conditions with the insulation placed in the air cavity (Table 7.16).

Reinforcement Stress

In all cases the column reinforcement is seen to be in compression even if the surrounding concrete is in tension. Because the concrete undergoes a major portion of its creep and shrinkage movements during the first year, a significant load is transferred to the reinforcement at a very early age (the steel stress at 1 year is up to about 70% of its stress at 10 years). Subsequent unloading of the column by the brick veneer reduces the steel stress (by 100 MPa in Table 7.13 at 10 years, comparing 4.0 and 0.0 mm expansion gaps) as does thermal contraction of the reinforcement during the winter, but never sufficiently to result in tensile forces in the reinforcement.

The maximum steel compressive stress is predicted in Table 7.13 to be 183 MPa after 10 years. The compressive stress arising strictly from application of the building loads in the reinforcement is only about 40 MPa. This has interesting implications to the designer since the introduction of bending moments in the column could result in compressive yielding of the steel long before expected. The maximum reinforcement compressive stress under moderate conditions is seen in Table 7.6 to be 141 MPa after 10 years.

Brick Veneer Stress

Figures 7.5 and 7.6 illustrate how the stress (negative values indicate compression) developed in the brick veneer at the ground level changes with time, under moderate and severe conditions respectively. Only those values corresponding to a shelf angle stiffness of 300 kN/mm/m and an insulation thickness of 25 mm are shown since, as will be shown later in this section, the ground level veneer stress is largely independent of shelf angle stiffness and insulation thickness. As would be expected, since most of the long term movements are hyperbolic in nature and tend to a limiting value, the axial stress developed in the veneer increases rapidly once contact is made with the upper shelf angle and gradually levels out. Under both conditions the stress at 1 year, for an expansion gap of zero, is about 70% of the 10 year value.

Figure 7.5 Variation in brick veneer compressive stress with time under moderate conditions and a shelf angle stiffness of 300 kN/mm/m

Figure 7.6 Variation in brick veneer compressive stress with time under severe conditions and a shelf angle stiffness of 300 kN/mm/m.

Figures 7.7 through 7.10 show the effect of expansion gap thickness on the stresses developed in the veneer, block infill, and the column concrete. The stresses have been plotted for two different times, 1 and 10 years from the beginning of construction, to illustrate the effect of time on the different curves. The following general observations can be made about all four figures;

- 1) The brick veneer and the concrete stress curves are shifted outwards (increasing compression in the veneer and increasing tension in the column concrete) with increasing time, but remain essentially parallel.
- 2) With increasing time, the block stress, for a given expansion gap, tends to zero. This is shown by an upwards shift in the block stress curve with time.
- 3) As the expansion gap thickness increases, the compressive stress developed in the brick veneer decreases and the compressive stress in both the column concrete and the block infill increases (as dictated by static equilibrium).
- 4) Once the brick veneer stress reaches zero, the stress in all elements becomes constant for increasing expansion gap thickness *at the time considered*. This is shown by the vertical tail at the top of each curve.
- 5) Once the blockwork comes into contact with the upper slab and begins to pick up some load, the stiffness of the frame is significantly increased and the compressive stress in the column concrete increases at a lesser rate for decreasing veneer stress. This phenomenon appears in Figures 7.7 to 7.10 as a kink in the concrete stress curve.

Table 7.21 summarizes the minimum expansion gap thicknesses required to ensure that the brick veneer remains unloaded during the first ten years of the building's life (for the various parameters considered in Figures 7.7 through 7.10).

Table 7.21 Minimum required expansion gap thickness (mm)

Condition	25 mm Insulation Location	
	Interior	Air Cavity
Moderate	2.75	2.90
Severe	3.65	3.80

Figure 7.8 Variation of exterior wall element stresses with expansion gap thickness under moderate conditions and 25 mm of insulation placed in the air cavity.

Figure 7.8 Variation of exterior wall element stresses with expansion gap thickness under moderate conditions and 25 mm of insulation placed in the air cavity.

Figure 7.9 Variation of exterior wall element stresses with expansion gap thickness under severe conditions and 25 mm of insulation placed on the interior face of the block and column.

Figure 7.10 Variation of exterior wall element stresses with expansion gap thickness under severe conditions and 25 mm of insulation placed in the air cavity.

The effect of insulation location on the compressive stress developed in the brick veneer at 10 years is shown in Figure 7.11. It can be seen that during the summer the stress increases by only 5 or 6% if the insulation is placed in the air cavity rather than on the interior face of the block and column. The effect of insulation location is much more dramatic during the winter, as can be seen by comparing Figure 7.12 with 7.13 and Figure 7.14 with 7.15. Placing the insulation in the air cavity, rather than on the interior face, results in a considerable decrease in the veneer compressive stress during the winter (varying from 35 to 50%).

Within each of the Figures 7.12 to 7.15, the effect of varying insulation thickness on the brick veneer stress is seen to be negligible during the summer, rising to only a 5 to 10% difference during the winter. In general, however, increasing the insulation thickness in the air cavity results in a decrease in the winter brick veneer stress, whereas increasing the insulation thickness on the interior face causes an increase in the winter brick veneer stress. The opposite is true during the summer to a lesser degree.

In all cases, the winter brick veneer stress is lower than the summer values and so it appears that the location and thickness of the insulation will not affect the probability of failure of the veneer except perhaps from a cyclic or fatigue point of view.

Figure 7.11 Variation of brick veneer stress with expansion gap thickness during the summer after 10 years.

Figure 7.13 Variation of brick veneer stress with expansion gap thickness after 10 years under moderate conditions and insulation placed on the interior face of the block and column.

Figure 7.13 Variation of brick veneer stress with expansion gap thickness after 10 years under moderate conditions and insulation placed in the air cavity.

Figure 7.14 Variation of brick veneer stress with expansion gap thickness after 10 years under severe conditions and insulation placed on the interior face of the block and column.

Figure 7.15 Variation of brick veneer stress with expansion gap thickness after 10 years under severe conditions and insulation placed in the air cavity

Figures 7 16 and 7 17 show the variation in ground level brick veneer stress with the shelf angle stiffness-yield parameters for zero expansion gap. The veneer stress is shown to be almost completely independent of the shelf angle stiffness. To understand this concept, it helps to consider an infinite series of shelf angles of any stiffness. For a unit growth in each of the supported brick panels, the accumulated axial stress developed in each of the panels, due to the deformation arising in each of the overlying shelf angles, tends to infinity unless the shelf angle deformation tends to zero. What this means is that the accumulated axial stress acting on any panel tends to the value required to reduce the shelf angle deformation to zero, implying in turn that the axial stress is only dependent on the magnitude of the unrestrained differential movement between the brick veneer and the frame.

It should be noted that for a small number of shelf angles the brick veneer stress begins to show some dependence on the shelf angle stiffness, as would be expected. Figures 7 18 to 7 21 show the variation in brick veneer stress with height for two different shelf angle stiffnesses, 300 and 500 $kN/mm/m$. It is only in the top 5 to 7 storeys that the brick veneer stress begins to show an increase with increasing shelf angle stiffness. The linear portions of these curves indicate yielding of the shelf angles.

A close inspection of Tables 7.17 to 7 20 shows that in fact the brick veneer stress actually decreases by a small amount with increasing shelf angle stiffness. To explain this, the early period in the building life must be considered, when only a few shelf angles are in contact with the underlying veneer. Under these conditions, larger shelf angle stiffnesses resulted in larger stresses, which in turn resulted in slightly larger creep strains. Thus the final unrestrained differential movement between the veneer and the frame will be slightly less for larger shelf angle stiffnesses, giving slightly smaller axial stresses when all of the shelf angles are acting.

When the yield force of each shelf angle becomes too small, the axial force required to return the unrestrained differential movement to zero may be impossible to develop with a finite number of angles. In this case the brick veneer axial stress is linearly dependent on the shelf angle yield force, which is represented in Figures 7 16 and 7 17 by the initial sloping line.

Figure 7.16 Variation of brick veneer compressive stress with shelf angle stiffness-yield parameters with 25 mm of insulation placed on the interior face of the block and column.

Figure 7.17 Variation of brick veneer compressive stress with shelf angle stiffness-yield parameters with 25 mm of insulation placed in the air cavity.

Figure 7.18 Variation of brick veneer stress with height after 10 years under moderate conditions, zero expansion gap, and 25 mm of insulation placed on the interior face of the block and column.

Figure 7.19 Variation of brick veneer stress with height after 10 years under moderate conditions, zero expansion gap, and 25 mm of insulation placed in the air cavity.

Figure 7.20 Variation of brick veneer stress with height after 10 years under severe conditions, zero expansion gap, and 25 mm of insulation placed on the interior face of the block and column.

Figure 7.21 Variation of brick veneer stress with height after 10 years under severe conditions, zero expansion gap, and 25 mm of insulation placed in the air cavity.

From the observations drawn from Figures 7.16 to 7.21, a simplified approach to the determination of the axial force acting on a lower level brick veneer panel, with no expansion gap provided, can be suggested as follows;

- 1) estimate the total unrestrained differential movement between the brick veneer and the frame, at the time considered,
- 2) calculate the axial force (acting in opposite directions on the brick veneer and the the frame) as the smaller of;
 - a) that force required to return the differential movement to zero, or
 - b) the sum of shelf angle yield forces above the level considered.

This procedure could be considered to be reasonably accurate if a sufficient number of shelf angles above the level considered are in contact with the brick veneer (about 5 to 10 depending on the angle stiffness).

In summary, the maximum brick veneer stresses were predicted under the following conditions;

- 1) at the ground level,
- 2) during the summer,
- 3) for an expansion gap thickness of zero,
- 4) at ten years, and
- 5) for the insulation placed in the air cavity.

Under moderate conditions, the maximum brick veneer compressive stress was predicted in Table 7.8 to be about 10.5 MPa, and under severe conditions Table 7.16 shows a value of 13.8 MPa. The ultimate compressive strength of the brick masonry can be estimated from Equation (3.64) to be 33.0 MPa, and from Equation (3.60) to be about 24.0 MPa. In either case, it appears likely that the compressive stresses arising from differential movement are not sufficient to cause failure in the veneer directly, particularly if the column concrete cracks in tension. However, distress or failure could easily arise from the following sources;

- 1) If the load is not transmitted evenly to the upper brick surface or if the joint at the toe of the shelf angle is mortared solid, stress concentrations could lead to local crushing or cracking.
- 2) If loading is eccentric, due to mortared toe joint or shelf angle rotation, the veneer may buckle, particularly if the lateral ties are missing in the region of the shelf angles.

Since the distress observed at the lower levels of the 18 storey structure considered here was typified by spalling and cracking rather than buckling, the stress concentration factors described in item (1) appear to be the major contributors to distress. Note that if the shelf angle extends $3/4$ of the way through the veneer and the toe is mortared solid, the effective local stress at the toe is increased to about 42 MPa under moderate conditions and 55 MPa under severe conditions. Both of these stresses are sufficient to initiate cracking and spalling. Subsequent cyclic and freeze-thaw action will result in a propagation of the distress with time.

Chapter 8

Conclusions and Recommendations

On the basis of this report, the following conclusions and recommendations can be made concerning the design of the veneer system on loadbearing masonry structures;

- 1) For the purposes of design, the maximum brick surface temperature should be assumed to be at least 55°C on a hot, clear summer day in southern Canada. For a north exposure, a reduced value of about 36°C may be used.
- 2) The annual temperature range experienced by the brick veneer should be assumed to be from 56°C for a north exposure, to 72°C for other exposures.
- 3) The location and, to a lesser extent, the efficiency of the insulation has a considerable effect on the annual cyclic differential movement between the veneer and the frame, but a negligible effect on the daily and long term differential movements. Placing the insulation in the air cavity results in the largest differential movements.
- 4) The ties between the brick veneer and the back-up blockwork should be designed for;
 - a) a long term differential movement of at least 1.6 mm per storey,
 - b) an annual cyclic differential movement of, at least 0.9 mm per storey (a reduced value may be used if the insulation is placed on the interior face of the blockwork and the blockwork is not restrained by interior walls),
 - c) a daily cyclic differential movement of at least 0.35 mm per storey

In the design of the ties, not only must the differential movement be allowed for, but the fatigue life of the system must also be considered. For example, if a tie is expected to last at least ten years, it must be designed to accommodate a long term movement of at least 1.6 mm per storey as well as ten cycles of at least 0.9 mm per storey and 3650 cycles of at least 0.35 mm per storey. Larger cavity widths can be used to increase the tie fatigue life.

- 5) Although in practice the presence of excessive amounts of mortar in the air cavity may act to reduce the observed differential movement between the veneer and the frame, this should not be considered in the design of the ties.

The following conclusions and recommendations can be made on the basis of this report concerning the design of the veneer system on reinforced concrete structures;

- 1) The width of the expansion gap provided under each shelf angle was found to have the greatest effect on the compressive stresses developed in the brick veneer. To allow for the differential movements occurring in the first 10 years of a building's life, the designer should provide an expansion gap of at least 3 mm per storey under each shelf angle if relatively moderate moisture movement and ambient thermal conditions are known to exist. If these conditions are not known, relatively severe values should be assumed in which case the provided expansion gap should be at least 4 mm per storey.
- 2) When the provided expansion gap is insufficient to allow for the differential movement, the brick veneer stress can be expected to increase rapidly at first, then gradually level out. If no expansion gap is provided, the compressive stress in the veneer reaches 70% of its 10 year value by the end of the first year. Thus it appears likely that if distress in the veneer is going to occur, it will occur quite early in the life of the building.
- 3) For structures with more than about 10 shelf angles over their heights, the axial compressive stress arising in the brick veneer at the lower levels was found to be largely independent of the shelf angle stiffness, being governed instead by the magnitude of the differential movement or the yield point of the overlying shelf angles. When fewer than about 10 shelf angles are acting above the level considered, the axial stress shows an increase with increasing shelf angle stiffness (to a maximum value defined by the magnitude of the differential movement).
- 4) Insulation thickness has a negligible effect on the stress developed in the brick veneer.
- 5) Insulation location has a negligible effect on the stress developed in the brick veneer during the summer and tends to reduce the stress during the winter if placed in the air cavity rather than the interior face of the block and column. However, the winter stresses are consistently lower than the summer stresses (for the construction timing assumed in Test Case Two) and so the insulation location will have a negligible effect on the occurrence of distress or failure in the brick veneer.
- 6) To avoid the possibility of distress or failure of the veneer, the designer should ensure that the following guidelines are incorporated into the design;
 - a) the joint at the toe of the shelf angle should be waterproofed with a suitable compressible sealant, but not mortared in,
 - b) metal ties should be provided and properly installed at regular intervals throughout the building height,
 - c) the brick should not overhang the shelf angle toe by more than 1/2 the brick width.
- 7) To minimize the differential movement between the brick veneer and the frame, the designer should;
 - a) store the brick and block as long as feasible, prior to their use, in a sheltered, but not arid, environment,

- b) provide a period as long as feasible between the placement of the concrete column and the brick veneer at all levels,
 - c) ensure that the shelf angle is adequately protected against corrosion (ie., provide a protective coating and a proper flashing detail),
 - d) use non-expansive mortar in the brick masonry.
- 8) In general the maximum axial compressive stress predicted in the brick veneer under the worst conditions did not exceed about $1/2$ the ultimate compressive strength of the masonry. However the effects of stress concentrations and eccentricity of the load could easily lead to distress under these conditions.

Recommendations for Future Work

In this report a relatively large number of assumptions have had to be made because of lack of information. In particular the following areas need to be researched in more detail,

- 1) The deformational properties of brick and block masonry are not at all well known. In particular, creep under incremental loading in both brick and block masonry, and the freezing expansion of clay brick need further illumination. Also the elastic properties of masonry are not well defined.
- 2) The thermal properties of building materials, in particular their response to solar radiation, need some further research.
- 3) The effect of mortar droppings between the veneer and its backup on differential movements has been assumed negligible in this report. However, recent observations on the Test Case One structure indicates that considerable restraint may in fact be present and the implications of this needs to be considered.
- 4) The effect of interior walls on the thermal movement experienced by exterior walls is at this moment unknown. A study of this phenomena should also include the stresses at the joints between exterior and interior walls.
- 5) Research is required to determine the fatigue life of various common masonry ties subjected to both cyclic and long term movements.
- 6) One observation made in this report, which was not followed up, concerns the transfer of stress to the column reinforcement. The implications that a high compressive stress in the reinforcement has on the structural performance of a column should be investigated.
- 7) In that the basis of this report is largely theoretical, further full-scale building experimentation should be carried out to verify the results presented here.

Appendix A

Material Thermal Properties

A.1 Clay Brick

A.1.1 Thermal Conductivity

For bone dry building materials, Jakob (53) gives thermal conductivity values as a function of the material density, from which the following relationship has been derived,

$$k_{dry} = \left(\frac{\rho}{2505.46} \right)^{6.642} + 2.5917 \times 10^{-4} \rho - 0.0504 \quad (A.1)$$

where,

k_{dry} = thermal conductivity of the bone dry building material, in $W/m^{\circ}C$

ρ = density of the building material in question in kg/m^3 (must lie between 500 and 2600 kg/m^3)

Jakob also provides a correction factor to account for the influence of the moisture in the material on its thermal conductivity, from which the following relationship has been derived;

$$f = 1 + 0.3157w^{0.6} - 0.01587w \quad (A.2)$$

where,

f = moisture content correction factor

w = moisture content in percent by volume

Figure A.1 shows Jakobs observed variation between thermal conductivity and density for bone dry brick and Table A.1 lists the values of thermal conductivity given by various other sources.

A.1.2 Specific Heat

Both Özisik (79) and Chapman (20) give the specific heat of clay brick as ranging from 840 to 960 $J/kg^{\circ}C$. Lewis et al (67) gives the specific heat of concrete brick to be 890 $J/kg^{\circ}C$.

A.1.3 Emissivity and Absorptivity

The total hemispherical emissivity is given by Grimm (39) to range from 0.85 to 0.95. Also given are absorptivity values for various brick surface colours, as follows;

- 1) White or light cream: $\alpha = 0.30$ to 0.50
- 2) Yellow or buff: $\alpha = 0.50$ to 0.70
- 3) Red or brown: $\alpha = 0.65$ to 0.85

Table A.1 Thermal conductivity of clay brick.

Brick Type	Density (kg/m^3)	Thermal Conductivity ($W/m^\circ C$)
Common (12,85)	1920	0.720
	1600	0.692
Face (12,85)	2080	1.296
	2050	1.315
Fire clay (20,79)	2320	1.004
	2640	1.004
Clay brick, 75% solid (36)	2420	1.227
Hollow brick (12)		
4 inch, 62.9% solid	2050	0.783
6 inch, 67.3% solid	2050	0.924
8 inch, 61.2% solid	2050	1.083
10 inch, 60.9% solid	2050	1.195
Hollow brick with vermiculite fill (12)		
4 inch, 62.9% solid	2050	0.524
6 inch, 67.3% solid	2050	0.570
8 inch, 61.2% solid	2050	0.599
10 inch, 60.9% solid	2050	0.605

Figure A.1 Thermal conductivity of dry brick (from Jakob, 53)

A.2 Air Cavity

A.2.1 Convection

For a mean air cavity temperature of 10 °C, Hutcheon (52) predicts that the convection coefficient in air cavities bounded on two sides can be approximated by;

$$hc_{10} = \left(\frac{1}{l}\right) \left[0.025 + 0.07909 \ln \left\{ 1 + \left(\frac{\Delta T \cdot l^3}{8.6097 \times 10^{-4}} \right)^{0.8} \right\} \right] \quad (A.3)$$

where,

hc_{10} = convection coefficient for a mean air gap temperature of 10 °C

l = width of the air cavity in metres

ΔT = temperature differential across the cavity in °C

For mean air temperatures other than 10 °C;

$$hc = hc_{10} \cdot \left[1 - 0.0018(T - 10) \right] \quad \text{for } l \cdot hc_{10} > 0.043 \quad (A.4)$$

$$hc = hc_{10} \quad \text{for } 0.029 \leq l \cdot hc_{10} \leq 0.043 \quad (A.5)$$

$$hc = hc_{10} \cdot \left[1 + 0.003(T - 10) \right] \quad \text{for } l \cdot hc_{10} < 0.029 \quad (A.6)$$

A.2.2 Radiation

Again from Hutcheon (52), the radiative heat flow coefficient, h_r , can be found as follows;

$$h_r = \sigma \cdot F_a \cdot F_e \cdot (T_1^2 + T_2^2)(T_1 + T_2) \quad (A.7)$$

where,

h_r = radiative heat flow coefficient in W/m^2C

$F_a = 1.0$ generally

$$F_e = \left(\frac{1}{\epsilon_1} + \frac{1}{\epsilon_2} - 1 \right)^{-1} \quad (A.8)$$

$\epsilon_{1,2}$ = emissivities of the two materials bounding the air cavity

$T_{1,2}$ = surface temperatures of the two materials bounding the air cavity
in °K

$$\begin{aligned}\sigma &= \text{Stefan-Boltzmann constant} \\ &= 5.67 \times 10^{-8}\end{aligned}$$

A.2.3 Conduction

The coefficient of thermal conductivity of air is usually negligible compared to the convective and radiative heat transfer coefficients, however this term does grow important for very narrow gap widths. The coefficient of thermal conductivity of dry air can be found from Eckert (31) as follows;

$$k = 0.00287 + 7.76 \times 10^{-5} \cdot T \quad (\text{A.9})$$

where,

k = coefficient of thermal conductivity of dry air in $W/m^{\circ}C$

T = mean temperature of the air in $^{\circ}K$

A.2.4 Specific Heat and Density

The density of dry air at $20^{\circ}C$ and 1 atmosphere is given by Chapman (20) to be $1.183 \text{ kg}/m^3$ and the specific heat (CP) is given by;

$$CP = 0.9169 + 2.577 \times 10^{-4} \cdot T - 3.9747 \times 10^{-8} \cdot T^2 \quad (\text{kJ}/\text{kg}^{\circ}K) \quad (\text{A.10})$$

A.3 Insulating and Wall Finish Materials

Material	Source (ref. no.)	Density ρ (kg/m ³)	Specific Heat (kJ/kg°C)	Emissivity ϵ	Absorptivity α	Thermal Conductivity (W/m°C)
Laminated asbestos felt	31	580	1.00	0.96	0.60	$3.964 \times 10^{-4}(T) + 0.0447$
146 laminations/m	31	580	1.00	0.96	0.60	$1.261 \times 10^{-4}(T) + 0.0178$
73 laminations/m	31	580	1.00	0.96	0.60	$1.622 \times 10^{-4}(T) + 0.0276$
Corrugated asbestos	31	580	1.00	0.96	0.60	$2.342 \times 10^{-4}(T) + 0.0142$
Rock wool	31	110-190	0.84			$1.261 \times 10^{-4}(T) + 0.0128$
Mineral fibre blankets	5	10-50	0.84			a
Cellular glass	5	140	1.00			$1.730 \times 10^{-4}(T) + 0.0058$
Glass fibre board	5	50-160	0.96			$1.614 \times 10^{-4}(T) - 0.0116$
Rigid polystyrene						
Extruded	5	30	1.21	0.90		$1.453 \times 10^{-4}(T) - 0.0063$
Molded beads	5	16	1.21	0.90		$1.297 \times 10^{-4}(T) + 0.0018$
Polyurethane	5	24-40	1.59	0.90	0.20	$0.0360 - 4.151 \times 10^{-5}(T)$
Loose fill						
Silica aerogel	5	125				$8.302 \times 10^{-5}(T)$
Vermiculite	5	64-96	13.4			$2.283 \times 10^{-4}(T) - 0.0047$
Vermiculite	5	110-130	13.4			$1.868 \times 10^{-4}(T) + 0.0126$
Perlite	5	80-130	1.09			$1.868 \times 10^{-4}(T) - 0.0019$

$$a) \quad k = 0.0047 - \frac{0.5895}{\rho} + \left(8.529 \times 10^{-5} + \frac{2.759 \times 10^{-3}}{\rho} \right) (T)$$

where T is the mean temp. of the material in °K

Insulating and Wall Finish Materials (cont'd)

Material	Source (ref. no.)	Density ρ (kg/m ³)	Specific Heat (kJ/m ² C)	Emissivity ϵ	Absorptivity α	Thermal Conductivity (W/m ² C)
Fibreglass bats	37	≥ 50	0.84			b
Fibreglass, preformed	37	40-100	0.96			c
Glass wool	37	16-80	0.84			$2.297 \times 10^{-4}(T) - 0.0310$
Rock wool	37	110-190	0.84			$2.715 \times 10^{-4}(T) - 0.0453$
Styrofoam	37	16-32	1.21	0.90	0.50	$1.686 \times 10^{-4}(T) - 0.0066$
Marble	37	2720	0.88			3.030
Gypsum board	37	800-1120	1.09	0.90	0.40	$2.036 \times 10^{-4}(T) - 0.0693$
Wood fibreboard	37	190-300	1.21	0.90	0.70	0.050
Mineral aggregate board	60	850	1.10			0.055
Softwood plywood, lumber, and partical board	60	540	1.20	0.90	0.70	0.115

$$b) \quad k = 0.0269 - \frac{2.1235}{\rho} + \left(1.0064 \times 10^{-5} + \frac{8.222 \times 10^{-3}}{\rho} \right) (T)$$

$$c) \quad k = 0.000165 - \frac{1.0571}{\rho} + \left(1.0274 \times 10^{-4} + \frac{4.488 \times 10^{-3}}{\rho} \right) (T)$$

A.4 Concrete Blockwork

A.4.1 Thermal Conductivity

The National Concrete Masonry Association (72, 74) has done a considerable amount of research into the coefficient of thermal conductivity of hollow block (assumed to be dry) of various sizes and densities. The effect of insulation placed in the cores was also investigated. The summarized results are shown in Table A.2 and plotted in Figure A.2. Based on Figure A.2, approximate relationships have been proposed between thermal conductivity and density for the various blocks tested, as shown in Table A.3.

NCMA also presents data for solid concrete units, again assumed to be dry, from which the following relationship has been derived;

$$k = 6.078 \times 10^{-7}(\rho)^2 - 1.131 \times 10^{-3}(\rho) + 0.7702 \quad (A.11)$$

where,

k = thermal conductivity of a solid concrete masonry unit in $W/m^{\circ}C$

ρ = density of the concrete in kg/m^3

Equation (A.11) is considered reliable for values of ρ between 960 and 2250 kg/m^3 .

The presence of moisture in the concrete has a significant effect on the thermal conductivity of the block. Making use of Jakob's (53) correction factor described in Section A.1;

$$f = 1.0 + 0.3157w^{0.6} - 0.1567w \quad (A.2)$$

where w is the moisture content in percent by volume, the coefficient of thermal conductivity of 'in-situ' blockwork can be estimated if the moisture content is known. Data presented by Sahlin (93) allows the approximation of the moisture content as a function of the mean relative humidity of the environment ($R.H.\%$);

$$w \simeq 0.0015(R.H.\%)^2 \quad (\%) \quad (A.12)$$

The presence of hollow cores, however, complicates the direct application of the moisture correction factor, since the presence of moisture really only effects the conductivity of the shell. Preliminary calculations of this thermal bridging effect indicates that only a portion of the correction factor is applicable to the blockwork as a whole, that portion being approximately equal to the percent solid of the hollow block unit used, ie.;

$$k_{tot} \simeq \left[\gamma(f - 1.0) + 1.0 \right] \cdot k_{totDRY} \quad (A.13)$$

where,

k_{tot} = net thermal conductivity of the block masonry in $W/m^{\circ}C$

k_{totDRY} = thermal conductivity of the dry block masonry as determined from Table A.2 and Equation (A.16), in $W/m^{\circ}C$

f = correction factor for the presence of moisture, as determined from Equation (A.2)

γ = fraction solid of the block masonry

$$= 1.0 - (1.0 - \alpha)(1.0 - \psi) \quad (A.14)$$

α = fraction of cores grouted

ψ = ratio of the net block to gross block unit area

Note that if insulation is present in the hollow cores, the coefficient γ should be replaced by γ_{ins} , where;

$$\gamma_{ins} \approx \frac{1}{2}(\gamma + 1.0) \quad (A.15)$$

If the wall is partially or completely grouted, the 'dry' thermal conductivity, k_{totDRY} , must be found from;

$$k_{totDRY} = (1 - \alpha) \cdot k_{hol} + \alpha \cdot k_{sol} \quad (A.16)$$

where,

k_{hol} = thermal conductivity of the hollow dry block (with or without insulation), as determined from Figure A.2 or Table A.2, in $W/m^{\circ}C$

k_{sol} = thermal conductivity of the solid dry block, as determined from Equation (A.11), in $W/m^{\circ}C$

Table A.2 Thermal conductivity values of hollow dry concrete block masonry, in $W/m^{\circ}C$ (from NCMA, 72).

Thickness (mm)	Insulation in cores	Density (kg/m^3)				
		900	1280	1600	1920	2240
100	filled	0.172	0.207	0.247	0.300	0.506
	empty	0.279	0.343	0.412	0.493	0.749
150	filled	0.155	0.188	0.232	0.293	0.544
	empty	0.384	0.473	0.565	0.670	1.006
200	filled	0.155	0.190	0.238	0.304	0.582
	empty	0.502	0.544	0.659	0.790	1.177
250	filled	0.154	0.193	0.243	0.314	0.613
	empty	0.480	0.601	0.732	0.884	1.335
300	filled	0.158	0.199	0.254	0.334	0.668
	empty	0.526	0.660	0.808	0.956	1.491

Table A.3 Proposed thermal conductivity relationships for hollow dry concrete block masonry.

Thickness (mm)	Insulation in cores	Thermal Conductivity ($W/m^{\circ}C$)	
		for $\rho \leq 1920 \text{ } kg/m^3$	for $\rho > 1920 \text{ } kg/m^3$
100	filled	$k = 1.333 \times 10^{-4}(\rho) + 0.044$	$k = 6.438 \times 10^{-4}(\rho) - 0.936$
	empty	$k = 2.229 \times 10^{-4}(\rho) + 0.065$	$k = 8.000 \times 10^{-4}(\rho) - 1.043$
150	filled	$k = 1.438 \times 10^{-4}(\rho) + 0.017$	$k = 7.844 \times 10^{-4}(\rho) - 1.213$
	empty	$k = 2.979 \times 10^{-4}(\rho) + 0.098$	$k = 1.050 \times 10^{-3}(\rho) - 1.346$
200	filled	$k = 1.552 \times 10^{-4}(\rho) + 0.006$	$k = 8.688 \times 10^{-4}(\rho) - 1.364$
	empty	$k = 3.625 \times 10^{-4}(\rho) + 0.094$	$k = 1.209 \times 10^{-3}(\rho) - 1.532$
250	filled	$k = 1.667 \times 10^{-4}(\rho) - 0.006$	$k = 9.344 \times 10^{-4}(\rho) - 1.480$
	empty	$k = 4.208 \times 10^{-4}(\rho) + 0.076$	$k = 1.409 \times 10^{-3}(\rho) - 1.822$
300	filled	$k = 1.833 \times 10^{-4}(\rho) - 0.018$	$k = 1.044 \times 10^{-3}(\rho) - 1.670$
	empty	$k = 4.479 \times 10^{-4}(\rho) + 0.096$	$k = 1.672 \times 10^{-3}(\rho) - 2.254$

Figure A.2 Thermal conductivity of hollow concrete block

Figure A.3 Correction factor for the thermal conductivity of solid concrete.

A.4.2 Specific Heat

ASHRAE (5) gives the specific heat of normal weight concrete block to be $0.92 \text{ kJ/kg}^\circ\text{C}$ and $0.88 \text{ kJ/kg}^\circ\text{C}$ for lightweight concrete block. Since the specific heat of water at 20°C is much larger than that of the block ($4.18 \text{ kJ/kg}^\circ\text{C}$), the presence of moisture in the block will effect its specific heat significantly.

$$CP_M = CP_{dry} \cdot \left(1 - \frac{w}{100}\right) + 4.178 \left(\frac{w}{100}\right) \quad (\text{kJ/kg}^\circ\text{C}) \quad (\text{A.17})$$

where,

CP_M = net specific heat of the concrete block

CP_{dry} = dry specific heat of the concrete block
 = $0.92 \text{ kJ/kg}^\circ\text{C}$ for normal weight block
 = $0.88 \text{ kJ/kg}^\circ\text{C}$ for lightweight block

w = moisture content in percent by volume

A.4.3 Emissivity and Absorptivity

Reagan (88) gives the absorptivity of plain grey concrete block with concave, low contrast mortar joints as 0.61. This value decreases to 0.55 if plain joints are used.

ASHRAE (5) gives a range of 0.85 to 0.95 for the emissivity of concrete block.

A.5 Concrete

A.5.1 Thermal Conductivity

The thermal conductivity of concrete, as presented by Neville (78) for various densities and two different moisture contents, is shown in Figure A.4. Jakob's (53) moisture content correction factor shows excellent agreement with Neville's observations, and so was used to determine the thermal conductivity of dry concrete, as shown in Figure A.5. For dry normal weight concrete, the dry thermal conductivity (k_{dry}) was found to relate to density as follows;

$$k_{dry} = -0.3703 + 5.0675 \times 10^{-4}(\rho) + \left(\frac{\rho}{2528.6}\right)^{0.249} \quad (W/m^{\circ}C) \quad (A.18)$$

Equation (A.18) is considered reliable for concrete densities, ρ , greater than about 1500 kg/m^3 .

The thermal conductivity of in-situ concrete can now be found from;

$$k = f \cdot k_{dry} \quad (W/m^{\circ}C) \quad (A.19)$$

where f is Jacob's correction factor as determined using Equation (A.2). The moisture content can be estimated to be 2.5% under protected conditions and 5.0% under exposed conditions.

A.5.2 Specific Heat, Emissivity, and Absorptivity

$$CP_{conc} = 0.92 \left(1 - \frac{w}{100}\right) + 4.178 \left(\frac{w}{100}\right) \quad (A.20)$$

Emissivity of Concrete = 0.85 to 0.95

Absorptivity of Concrete = 0.55 to 0.65

Figure A.4 Thermal conductivity of normal weight concrete

Figure A.5 Thermal conductivity of dry concrete

Appendix B

Effective Thickness of Projecting Concrete Column

As noted in Chapter 2, the concrete column, with one face flush against the exterior wall, acts as a thermal fin, with heat loss occurring from the remaining three surfaces. To find the average temperature of the column using a one-dimensional heat flow model, an estimation of the column's effective thickness must be made. This so called effective thickness (b_{EFF}) depends on many different parameters, of which the most important were judged to be,

- 1) The width of the column measured parallel to the face of the wall. As the column width increases, the effective thickness approaches the true thickness
- 2) The true thickness of the column projecting beyond the wall face. As this value increases, the effective thickness increases to a limiting value
- 3) The thermal resistance of the exterior wall. As the thermal resistance of the wall increases, the effective thickness of the column approaches its true thickness (although the variation is not great over the typical range of thermal resistances encountered in practice)

To determine how the effective thickness varies with these parameters, the column temperatures derived from a two-dimensional steady state heat flow model were compared with those derived from a one-dimensional steady state analysis. While the true column dimensions were used in the two-dimensional analysis, varying values of effective thickness were applied to the one-dimensional case. The column was assumed to be rectangular throughout.

Using the finite element mesh shown in Figure B 1, the temperature distribution in the projecting fin typically appeared as shown in Figure B 2, for the one-dimensional heat flow analysis

Figure B.1 Finite element mesh and numbering system used for the one-dimensional heat flow analysis

Figure B.2 Temperature distribution through the concrete column elements, F and G, as determined by the one-dimensional heat flow analysis.

Both the one and the two-dimensional heat flow analyses were performed using a finite element transient heat flow computer program developed by the Mechanical Engineering Department at Carleton University, Ottawa, Canada. To simulate steady state conditions, the duration of the heat flow was set to 40 000 seconds. Even over such a large period of time, some non-linear variation in temperature along the thickness of the column (in the y direction shown in Figure B.2) was observed in the one-dimensional analysis (it should be linear for a constant thermal conductivity). The discrepancy, however, is small and so all of the heat flow calculations were based on a heat flow time of 40 000 seconds.

Using Simpson's rule, the average temperature of the projecting fin in the one-dimensional analysis can be found to be;

$$T_{ave} = \frac{1}{12}(T_{11} + 4T_9 + 2T_6 + 4T_4 + T_1) \quad (^\circ C) \quad (B.1)$$

where T_i represents the temperature at node i , which is constant in the x direction, and T_{ave} is the average temperature of the column.

Similarly, a finite element mesh, shown in Figure B.3, was set up for the two-dimensional heat flow analysis, and a typical temperature profile is shown in Figure B.4 for half of the projecting column. To illustrate the fact that the temperature of the projecting portion of the column is much closer to the interior temperature than would be predicted by a one-dimensional analysis,

the temperature contours for a homogeneous half wall are plotted in Figure B.5. The average temperature of the column can again be found by applying Simpson's rule to Figure B.4 or B.5, performing the summation first in the x direction and then in the y direction.

In the two-dimensional analysis, the column dimensions, width and thickness, were varied from 100 mm to 800 mm in five increments (100, 200, 300, 500, and 800 mm), whereas the effective thickness of the column in the one-dimensional analysis was varied from 25 to 250 mm in 25 mm increments. Only two values of wall thermal resistance were considered, one being typical for cavity walls found in North America ($R = 1.13 \text{ m}^2 \cdot ^\circ\text{C}/\text{W}$), the other being more representative of an uninsulated wall ($R = 0.24 \text{ m}^2 \cdot ^\circ\text{C}/\text{W}$). In all cases the interior temperature was set to 0°C and the exterior to 10°C . These values are entirely arbitrary and, while they are reflected in the average column temperatures, they do not affect the determination of the effective thickness, b_{EFF} .

The average column temperature, as calculated in the one-dimensional analysis, versus the effective thickness of the column is shown in Figure B.6. Once the average column temperatures, yielded by the two-dimensional analysis for the various column dimensions, have been found, Figure B.6 can be entered to pick the required effective thicknesses.

Figure B.7 shows the variation of the effective thickness with the column width for various values of the true column thickness (for a wall resistance $R = 1.13 \text{ m}^2 \cdot ^\circ\text{C}/\text{W}$). The curves show marked divergence only for column widths above about 420 mm. The following relationships show reasonably good agreement with the observed data for a wall thermal resistance of $1.13 \text{ m}^2 \cdot ^\circ\text{C}/\text{W}$;

$$b_{EFF, 1.13} = 0.209d - 16.0 \quad \text{for } 150 \text{ mm} \leq d \leq 420 \text{ mm} \quad (B.2)$$

$$= m(d - 420) + 70 \quad \text{for } d > 420 \text{ mm} \quad (B.3)$$

where,

$b_{EFF, 1.13}$ = effective thickness of the projecting column, for a wall resistance of $1.13 \text{ m}^2 \cdot ^\circ\text{C}/\text{W}$, in mm

d = true width of the column, measured parallel to the wall face, in mm

$$m = 1.0774 \times 10^{-9}b^3 - 2.018 \times 10^{-6}b^2 + 1.305 \times 10^{-3}b - 0.1043 \quad (B.4)$$

b = true thickness of the column projecting beyond the wall face in mm

Since only two column sizes (300×300 and 500×500 mm) were run in the two-dimensional analysis at the lower wall resistance ($R = 0.24 \text{ m}^2 \cdot ^\circ\text{C}/\text{W}$), the following correction factor, for wall resistances other than $1.13 \text{ m}^2 \cdot ^\circ\text{C}/\text{W}$, can only be considered as a rough estimate. If the corrected effective thickness is defined as;

$$b_{EFF} = b_{EFF,1.13} + C \quad (B.5)$$

in mm, then the correction factor C can be found as shown in Figure B.8;

$$C = (R - 1.13)(0.0128b_{EFF,1.13} - 0.301) \quad \text{mm} \quad (B.6)$$

where R is the true wall thermal resistance in $\text{m}^2 \cdot ^\circ\text{C}/\text{W}$.

Figure B.4 Temperature profile of half the projecting column from the two-dimensional heat flow analysis.

Figure B.5 Heat flow through the wall-column section.

Figure B.6 Average column temperature predicted by the one-dimensional heat flow analysis (from Eq. (B.1)).

Figure B.7 Variation of effective thickness, b_{eff} , with column width, d , for a wall thermal resistance of $1.13 \text{ m}^2 \cdot ^\circ \text{C/W}$.

Figure B.8 Variation of effective thickness correction factor with wall thermal resistance.

Appendix C

Solar Radiation

To allow a reasonable estimate of the extreme temperatures which arise in the veneer of a building, it is necessary to take into account the effect of solar radiation. The brick surface temperature is often as much as 30 C° above the ambient air temperature due to solar radiation. The energy incident on a vertical surface will be discussed in this appendix in detail, as derived from ASHRAE (5)

The amount of incident solar radiation on a surface is dependent upon,

- 1) time of day,
- 2) location on the surface of the planet,
- 3) orientation of the surface considered,
- 4) clearness of the sky

For the purposes of this report, the surface considered will be assumed to be vertical at all times with 'orientation' referring to the azimuth of the surface normal.

The mean magnitude of the solar radiation (I'_0) at the mean Earth-Sun distance from the sun is;

$$I'_0 = 1.37 \text{ kW/m}^2 \quad (\pm 0.7\%) \quad (C.1)$$

Since the Earth orbits in an ellipse around the sun, the radiation received at the Earth's surface will vary with the time of year;

$$I_0 \approx I'_0 \left[1 - 0.0167 \cos \{0.0172(DN - 3)\} \right]^{-2} \quad (C.2)$$

where,

I_0 = radiant energy incident on the Earth's surface at noon at 45° latitude, in kW/m²

DN = day number of the year (for example, on Jan. 1, $DN = 1$, on July 1, $DN = 182$, and on Dec. 31, $DN = 365$)

Note that all calculations are performed using Radians unless otherwise specified

The real radiant energy received at the surface of the Earth is considerably attenuated by the atmosphere. A horizontal surface with the sun directly overhead would receive,

$$A = 1.1589 + 0.07515 \cos \{0.0172(DN - 3)\} \text{ kW/m}^2 \quad (C.3)$$

As the sun moves from the overhead position, its radiation must pass through greater thicknesses of atmosphere. The resulting reduced solar radiation can be found from;

$$I_{DN} = \frac{A \cdot C_l}{e^{(B/\sin \beta)}} \quad (C.4)$$

where,

I_{DN} = direct normal solar radiation incident on a horizontal surface in kW/m^2

A = apparent solar radiation for an air mass of 1.0 (ie., the sun directly overhead at sea level) in kW/m^2

B = atmospheric extinction coefficient
 $\simeq 0.17405 - 0.03325 \cos \{0.0172(DN - 3)\}$ (C.5)

β = altitude of the sun in Radians above the horizon

C_l = clearness coefficient for the region (see ASHRAE, 5)
 $= 1.05$ for Ottawa, Canada

The altitude of the sun can be determined from;

$$\sin \beta = \cos(L) \cdot \cos(\delta) \cdot \cos(H) + \sin(L) \cdot \sin(\delta) \quad (C.6)$$

where,

L = local latitude in Radians

δ = solar declination in Radians
 $\simeq -0.4091 \cos \{0.0172(DN - 9.25)\}$ (C.7)

H = hour angle
 $= 4.3633 \times 10^{-3}(AST - 720)$ (C.8)

AST = apparent solar time in minutes

The apparent solar time can be related to local standard time as follows;

$$AST = LST + ET + 4(LSM - LON) \quad (C.9)$$

where,

LST = local standard time in minutes from midnight

ET = equation of time in minutes (ie., the variation between mean time and apparent solar time)

$$\simeq -7.4 \sin(0.0172DN) - 9.8 \sin(0.4 + 0.0344DN) \quad (C.10)$$

LSM = local standard time meridian in degrees of arc

- = 60° for Atlantic Standard Time
- = 75° for Eastern Standard Time
- = 90° for Central Standard Time
- = 105° for Mountain Standard Time
- = 120° for Pacific Standard Time
- = 135° for Yukon Standard Time
- = 150° for Alaska-Hawaii Standard Time

LON = local longitude in degrees of arc

For a vertical wall, the angle of incidence, θ , of the radiation striking the wall can be found from;

$$\cos(\theta) = \cos(\beta) \cdot \cos(\gamma) \quad (C.11)$$

where γ is the horizontal angle between the wall normal and the sun in Radians. To determine γ , the solar azimuth, α , must first be determined (measured in Radians clockwise from south) as follows;

$$\sin(\alpha) = \frac{\cos(\delta) \cdot \sin(H)}{\cos(\beta)} \quad (C.12)$$

If the azimuth of the wall normal, measured clockwise from north, is defined as ψ , then the horizontal angle between the wall normal and the sun can be found from;

$$\gamma = \alpha + \pi - \psi \quad \text{Radians} \quad (C.13)$$

Note that if $|\gamma| \geq \frac{\pi}{2}$ Radians, then the wall is in shade.

The effective solar radiant energy, I_{DR} , striking the surface of the wall can now be found to be;

$$I_{DR} = I_{DN} \cdot \cos(\theta) \quad kW/m^2 \quad (C.14)$$

Diffuse Solar Radiation

Along with the direct solar radiation, the surface considered will also receive a significant amount of radiant energy reflected from the foreground and from the atmosphere. This form of

reflected radiant energy is called diffuse solar radiation. The diffuse solar radiation falling on a vertical surface from a clear sky is given approximately by;

$$I_{DS} = C_S \cdot F_{SS} \cdot I_{DN} \quad kW/m^2 \quad (C.15)$$

where,

C_S = diffuse radiation factor for a clear sky

F_{SS} = surface to sky view factor

= 0.5 for a vertical surface facing a horizontal foreground

I_{DN} = direct normal solar radiation in kW/m^2 (see Eq. (C.4))

The diffuse radiation factor C_S is dependent on the time of the year and can be found approximately from;

$$C_S \simeq 0.0989 - 0.0399 \cos(0.0172DN - 0.1612) - \frac{DN}{4000} \left[\cos \left(\frac{0.0172DN - 0.1612}{2} \right) \sin \left(\frac{0.0172DN - 0.1612}{2} \right) \right]^2 \quad (C.16)$$

The diffuse solar radiation striking a vertical surface from the foreground can be determined as follows;

- 1) The total solar radiation falling on the ground (I_{TG}) is given by;

$$I_{TG} = I_{DN} [C_S + \sin(\beta)] \quad kW/m^2 \quad (C.17)$$

- 2) The ground reflected diffuse radiation incident on the surface in question (I_{DG}) is thus;

$$I_{DG} = C_G \cdot F_{SG} \cdot I_{TG} \quad kW/m^2 \quad (C.18)$$

where,

C_G = reflectance coefficient of the foreground

= 0.10 for a parking lot

= 0.20 for gravel or old concrete

= 0.25 for bright green grass

F_{SG} = surface to ground view factor

= $1 - F_{SS}$

(C.1

The total radiant energy incident on the vertical surface in question (G_{sol}) can now be calculated to be;

$$G_{sol} = 1000(I_{DR} + I_{DS} + I_{DG}) \quad W/m^2 \quad (C.20)$$

Appendix D

Comparison of Proposed Creep Calculations for Blockwork with Observations by NCMA

NCMA (76) performed creep and shrinkage tests on grout and block prisms (ungrouted) in which the age of the specimen upon loading was varied from 7 to 180 days. In all cases, the specimen age when measurements were discontinued was about 360 days. Although not specifically stated, it is assumed that the ambient conditions were controlled at about 21 °C and 50% relative humidity.

The creep prediction parameters, as derived in Section 3.4.1, are as follows;

$$\begin{aligned}\lambda &= 0.98 + 4.0478 \times 10^{-4} e^{0.10226(R.H.\%)^2} \\ &= 1.047 \text{ for 50\% relative humidity}\end{aligned}$$

$$\begin{aligned}d_{mBL} &= \lambda \cdot \frac{2A_g}{P} \\ &= 89.8 \text{ mm for the } 150 \times 200 \text{ mm block prisms}\end{aligned}$$

$$d_{mGR} = 39.3 \text{ mm for the } 75 \times 75 \text{ mm grout specimens}$$

$$\begin{aligned}\psi_f &= \left[4.1034 \left(1 - \frac{R.H.\%}{100} \right)^{0.613} \right] \left[5.675 \left(\frac{1}{d_m} \right)^{0.05} - 2.8045 \right] \\ &= 4.636 \text{ for the block prisms} \\ &= 5.149 \text{ for the grout specimens}\end{aligned}$$

The creep coefficient Φ can be found from;

$$\Phi(t, t_0) = \beta_a(t_0) + 0.4\beta_d(t - t_0) + \psi_f[\beta_f(t) - \beta_f(t_0)]$$

where the terms β_a , β_d , and β_f are defined in Section 3.4.1, t represents the age of the specimen at the time of interest (in this case 360 days), and t_0 represents the age of the specimen at the time of loading (7, 28, 90, and 180 days). The applied load is constant and so the total creep strain at any time can be found from;

$$\begin{aligned}\epsilon_{cr}(t) &= \frac{\sigma}{E(28)} \cdot \Phi(t, t_0) \\ &= \epsilon_{el}(28) \cdot \Phi(t, t_0)\end{aligned}$$

in which $\epsilon_{el}(28)$ is the elastic strain observed for loading at 28 days;

$$\epsilon_{el}(28) = 269 \times 10^{-6} \text{ mm/mm for the grout specimens}$$

$$\epsilon_{el}(28) = 145 \times 10^{-6} \text{ mm/mm for the block prisms}$$

The creep strains predicted for the grout show very good agreement with values observed by NCMA, as seen in Table D.1. However, in Table D.2, the strains predicted for the block prisms show poor agreement with NCMA observations. In this case it was assumed that the age of the block units, at the time of construction of the prisms, was zero, in other words the block units and the block prisms were assumed to be created simultaneously. Although NCMA does not give the age of the block units at the time of construction, if a reasonable value of 60 days is used the predicted creep strains show much better agreement with NCMA, as seen in Table D.3.

Table D.1 Comparison of predicted creep strains with those observed by NCMA (76) on grout specimens.

t_0 (days)	$\beta_a(t_0)$	$0.4\beta_a(t - t_0)$	$\beta_f(t)$	$\beta_f(t_0)$	$\Phi(t, t_0)$	Predicted Creep Strain ($\mu\text{m}/\text{m}$)	Observed Creep Strain ($\mu\text{m}/\text{m}$)	Percent Error
7	0.4587	0.3558	0.8240	0.2341	3.8495	1035	1244	-20
28	0.2537	0.3526	0.8240	0.4340	2.6144	703	763	-9
90	0.1252	0.3410	0.8240	0.6202	1.5157	408	396	3
180	0.0748	0.3165	0.8240	0.7276	0.8877	239	267	-12

Table D.2 Comparison of predicted creep strains with those observed by NCMA (76) on block prisms.

t_0 (days)	$\beta_a(t_0)$	$0.4\beta_a(t - t_0)$	$\beta_f(t)$	$\beta_f(t_0)$	$\Phi(t, t_0)$	Predicted Creep Strain ($\mu\text{m}/\text{m}$)	Observed Creep Strain ($\mu\text{m}/\text{m}$)	Percent Error
7	0.4587	0.3558	0.7868	0.2173	3.4524	501	248	-50
28	0.2537	0.3526	0.7868	0.4004	2.3974	348	199	-43
90	0.1252	0.3410	0.7868	0.5817	1.4170	206	168	-18
180	0.0748	0.3165	0.7868	0.6879	0.8499	123	137	11

Table D.3 Comparison of predicted creep strains with those observed by NCMA (76) on block prisms with block 60 days old prior to construction.

t_0 (days)	$\beta_a(t_0)$	$0.4\beta_a(t - t_0)$	$\beta_f(t)$	$\beta_f(t_0)$	$\Phi(t, t_0)$	Predicted Creep Strain ($\mu\text{m}/\text{m}$)	Observed Creep Strain ($\mu\text{m}/\text{m}$)	Percent Error
67	0.1527	0.3558	0.8070	0.5356	1.7667	256	248	-3
88	0.1272	0.3526	0.8070	0.5782	1.5405	223	199	-11
150	0.0862	0.3410	0.8070	0.6604	1.1067	161	168	4
240	0.0594	0.3165	0.8070	0.7302	0.7318	106	137	29

Note that the age of the blockwork prisms in Table D 3, at the time of interest, becomes 420 days for block 60 days old prior to construction.

In summary, it is recommended that the true age of the block be used when calculating the creep strain in ungrouted blockwork. If the age is not known, the block should be assumed to be 60 days old prior to placement since this gives creep strains in reasonable agreement with NCMA observations.

Appendix E

Computer Program User's Manual

E.1 Introduction

The computer program developed for this report models the long term and instantaneous vertical movements that occur in highrise structures and derives from these movements an estimation of the total axial stress in each of the exterior structural wall elements. The limitations of the program and the assumptions made to simplify the analysis are described in detail in Chapter 5. At the moment the program VENEER is operational on the CP-V Honeywell computer system at Carleton University, Ottawa, Canada. The source program was written in a low-level Fortran (Fortran IV) primarily because of its compatibility with many other systems.

The user's manual described herein is basically a guide to the user on the data input procedure. Since in many cases a particular building parameter can be specified in several different ways, the manual discusses the various alternatives, the procedure required for each, and the default values supplied. Unfortunately the program is not particularly user friendly (even to the author) and so it is recommended that the user follow the instructions with care, particularly with regard to the number of data lines required to specify a particular parameter.

Note: The program assumes that the building storeys are numbered sequentially with the lowest basement level being number one. The user must renumber his building bearing in mind that data specified for level 1 refers to the lowest basement, level 2 to the next higher level, etc.

Data Input Procedure

- 1) Case Title, FORMAT(20A4) on the first line of the data file input a case title of 80 characters or less
- 2) Job Control Data, FORMAT(10L2) on a new line, input 10 logical values (ie , T or F) separated by a blank, each of which indicate the following if true (T) respectively,
 - i) summer transient thermal analysis is to be performed
 - ii) input data is to be printed out
 - iii) debug data from subroutine RDATA is to be printed out
 - iv) loading history values for each level are to be printed out
 - v) the initial temperature of each wall element is to be printed out
 - vi) effective column thickness values are to be printed out
 - vii) effective shelf angle gap values and shelf angle stiffness parameters are to be printed out
 - viii) solar radiation debug data is to be printed out
 - ix) max/min temperature and unrestrained movement data from the transient heat flow analysis is to be printed out
 - x) shelf angle shear force analysis is not to be performed (ie , the shelf angle stiffness is negligible)

The default for any non-prescribed value is false (F)

- 3) Shelf Angle Shear Analysis Debug Data, FORMAT(2L2,2I2,L2,2I2): on a new line, input a series of control values to indicate the type of shear analysis debug data required. Note that this is only useful to someone with a working knowledge of the program. The sequence and meaning of each of the control values are as follows;
 - i) Logical Flag (L2): input 'T' if shear analysis debug data is desired for 'extreme' conditions. Default is no output.
 - ii) Logical Flag (L2): input 'T' if shear analysis debug data is desired for average conditions between the analysis intervals indicated by the following two integers (average condition stresses are also output for the time intervals specified). Default is no output.
 - iii) Two Integers (2I2): input two numbers corresponding to the internal time interval numbers between which, and including, the shear analysis debug data is to be output. Note that because of the large volume of data generated, it is desirable to minimize the number of intervals for which data is output

- iv) Logical Flag (L2): input 'T' if average condition stresses are desired between the time intervals specified by the following two integers. Default is no output.
 - v) Two Integers (2I2): input two numbers corresponding to the internal time interval numbers between which, and including, the average condition stresses are to be output.
- 4) Basement Heat Flow Data, FORMAT(L2,2I2): on a new line, input 3 control values to indicate the type of basement heat flow data desired. The sequence and meaning of each of the values are as follows;
- i) Logical Flag (L2): input 'T' if basement heat flow data is desired. Default is no output.
 - ii) Integer (I2): input a number corresponding to the level at which the basement heat flow data is desired (must be a basement level).
 - iii) Integer (I2): input a number to indicate the cross-section at which the basement heat flow data is desired, ie., input;
 - 0 if both midbay and column cross-section data is desired,
 - 1 if midbay cross-section data only is desired,
 - 2 if column cross-section data only is desired.
- 5) Steady State Heat Flow Data, FORMAT(L2,7I2): on a new line, input a series of control values to indicate the type of steady state heat flow data desired. The sequence and meaning of the values are as follows;
- i) Logical Flag (L2): input 'T' if steady state heat flow data is to be printed out. Default is no output.
 - ii) Integer (I2): input a number corresponding to the level at which the steady state heat flow data is desired (must be above ground).
 - iii) Integer (I2): input a number to indicate the cross-section at which the heat flow data is desired (see step (4) (iii)).
 - iv) Five Integers (5I2): input up to five integers to indicate the thermal conditions under which the heat flow data is desired, ie., input;
 - 1 for ambient temperature equal to the average expected air temperature,
 - 2 for ambient temperature equal to the maximum summer afternoon temperature (only performed if the transient analysis is not),
 - 3 for ambient temperature equal to the minimum winter temperature (only performed if a winter 'time of interest' is specified),
 - 4 for ambient temperature equal to the average annual temperature,

5 for ambient temperature equal to the summer dawn temperature (only performed if the transient analysis is performed).

6) Transient Heat Flow Data, FORMAT(2L2,2I2): on a new line, input a series of control values to indicate the type of transient heat flow data desired. The sequence and meaning of the control values are as follows;

- i) Logical Flag (L2): input 'T' if temperature values for each layer in the wall sandwich over a 24 hour period are to be printed out for the level specified in item iii). Default is no output.
- ii) Logical Flag (L2): input 'T' if the temperatures of each of the finite elements (in $^{\circ}K$) of the wall section over a 24 hour period are desired for the level and cross-section specified in items (iii) and (iv). Default is no output.
- iii) Integer (I2): input a number corresponding to the lowest unique level, which has the parameters of interest, at which the transient heat flow data is to be printed out. For example, if the designer is interested in the temperatures at level 6 but a lower level is identical, then the lower level should be specified here.
- iv) Integer (I2): input a number to indicate the cross-section at which the transient heat flow data is desired (0 for both, 1 for midbay only, and 2 for column section only, as in step (4) (iii)).

7) Frame Type, FORMAT(2A4): on a new line, input a series of characters to indicate the type of building frame considered, starting in column 1. The choices are as follows;

- i) CONCRETE
- ii) STEEL
- iii) MASONRY

8) Number of Storeys and Basements, FORMAT(2I5): on a new line, input two integers indicating the number of storeys above ground and the number of basement levels in the structure, respectively.

9) Storey Height, FORMAT(5(I2,F8.2)): on a new line, input up to five data pairs, each pair indicating the level number considered and its height in metres, respectively, for;

- i) level one (ie., the lowest basement),
- ii) any level at which a change in storey height occurs as one rises up the building (default is equal to the next lower specified value).

Continue for as many lines as necessary, working from the bottom to the top.

10) Tributary Area, FORMAT(5(I2,F8.2)): on a new line, input up to five data pairs each indicating the level number and its ceiling tributary area in m^2 , respectively, for;

- i) level one,

- ii) any level at which a change in ceiling tributary area occurs (default is the next lower specified value).

Continue for as many lines as necessary, working from the bottom to the top of the building.

- 11) Dead Loads, FORMAT(5(I2,F8.2)): on a new line, input up to five data pairs each indicating the level number considered and the dead load, in kN/m^2 , acting on its tributary area, respectively, for;

- i) level one,
- ii) any level at which a change in dead load occurs (default is the next lower specified value).

Continue for as many lines as necessary, working from the bottom to the top of the building.

- 12) Live Loads, FORMAT(5(I2,F8.2)): on a new line, follow the procedure outlined for dead loads (input live loads).

- 13) Blank Line: provide a blank line to indicate the end of this portion of the data file.

- 14) Live Load Contribution, FORMAT(2F10.3): on a new line, input two numbers, as fractions of 1, indicating the portion of the live load contributing to creep in the structure and the portion assumed to be acting simultaneously with 'extreme' thermal conditions, respectively. Default values are 0.20 and 0.50, respectively.

- 15) Building Location, FORMAT(A5,2F10.3): on a new line, input three data values described as follows, respectively;

- i) Local Standard Meridian (A5): enter EST for Eastern Standard Time, AST for Atlantic Standard Time, AHST for Alaska-Hawaii Standard Time, etc., starting in column 1.
- ii) Latitude (F10.3): input latitude in degrees, using negative values for south latitudes.
- iii) Longitude (F10.3): input longitude in degrees.

Default location is Ottawa, Canada (ie., EST, 45.3°, 75.7°, respectively).

- 16) Thermal Parameters, FORMAT(2F10.3,I5,2X,A3,F10.3): on a new line, input five data values described as follows, respectively;

- i) sky clearness coefficient for the location considered (from ASHRAE, 5). Default value is 1.05. (F10.3)
- ii) azimuth of wall normal in degrees clockwise from north. Default value is 0°. (F10.3)
- iii) wind exposure condition code (from NBC, 71), (I5), ie., input;
 - 1 for exposed location (default)
 - 2 for urban location

3 for downtown location

- iv) input the character string 'AIR' if the building is airconditioned. Default is non-airconditioned. (2X,A3)
 - v) input the average interior air temperature in °C. Default is 20 °C. (F10.3)
- 17) Thermal Parameters, FORMAT(2F10.3): on a new line, input the following two data values;
- i) average annual ambient temperature in °C. Input 999.0 for the default value of 5.8 °C. (F10.3)
 - ii) average annual ambient temperature range (from average minimum to average maximum) in C°. Default is 31.6 C° (if default is specified in item (i)). (F10.3)
- 18) Thermal Parameters, FORMAT(3F10.3): on a new line, input the following three data values;
- i) maximum summer afternoon ambient temperature in °C. Input 999.0 for a default value of 34.2 °C. (F10.3)
 - ii) corresponding ambient temperature at dawn of the same day in °C. Default is 20.1 °C (if default is specified in item (i)). (F10.3)
 - iii) minimum winter ambient temperature in °C. Default is -31.0 °C (if default is specified in item (i)). (F10.3)
- 19) Wind and Relative Humidity, FORMAT(2F10.3): on a new line, input the following two data values;
- i) mean annual wind speed in *m/sec*. If the entered value is less than zero, the wind-speed defaults to 4.2 *m/sec*. (F10.3)
 - ii) mean annual relative humidity in percent. If the entered value is less than zero, the relative humidity defaults to 75%. (F10.3)
- 20) Daily Windspeed Variation, FORMAT(I5,6F10.3): on a new line, input up to seven data values described as follows, respectively;
- i) control code (I5), input;
 - 0 or blank if daily windspeed is constant at the first value specified in item (ii).
 - 1 if daily windspeed variation is going to be specified in item (ii).
 - 2 if daily windspeed is constant and equal to the mean annual windspeed.
 - ii) input up to 6 data values (6F10.3), depending on the control code specified, indicating the average windspeed during each 4 hour period (from midnight to the next midnight) of the day for which 'extreme' thermal conditions are to be applied, in *m/sec*.
- 21) Relative Humidity Variation, FORMAT(I5,4F10.3): on a new line, input up to five data values described as follows, respectively;

i) control code (15), input;

0 or blank if seasonal relative humidities are identical to the mean annual relative humidity.

1 if the seasonal relative humidities are to be specified in item (ii).

ii) if the specified control code is 1, input four average seasonal relative humidities (4F10.3) in percent for winter (Jan., Feb., Mar.), spring (Apr., May, June), summer (July, Aug., Sept.), and fall (Oct., Nov., Dec.), respectively.

22) Window Condition, FORMAT(A4,6X,15I3): on a new line, input up to 16 data values to indicate whether the windows are open or closed at each level on the day that the 'extreme' thermal conditions are to be applied. The required data values are described as follows, respectively;

i) input either OPEN or CLOSED (A4,6X), default is OPEN, followed by,

ii) input up to 15 level numbers at which the above condition is true (15I3).

Note that since there may be more than 15 levels altogether, it is important to specify in item (i) whichever condition occurs the least number of times. For example, if all windows are closed, input a blank line. This implies that no windows are open.

23) Vertical Shading Angles, FORMAT(15I5): on a new line, input the effective angular position of the horizon, as seen by each level, in degrees above the horizontal. Input a value or blank for each level (leaving blanks for basement levels) in the building (15I5) and continue for as many lines as necessary for all levels, working from the bottom to the top. Default is 0°

24) Horizontal Shading Angles, FORMAT(6I5,5X,10I3): on a new line, and continuing for as many lines as necessary, input the following;

i) in degrees, measured clockwise from the left wall face (looking out), input the horizontal angular positions of shading obstructions (up to three allowed) as seen from the midpoint of the wall. Input the angle that the obstruction is seen to start followed by the angle that the obstruction is seen to end. Repeat for three obstructions, leaving blanks if less than three exist (6I5). A blank is assumed to be 0°. Note that the defining angles are in pairs and the second angle in each pair must be greater than (or equal) to the first.

ii) specify the levels at which the above shading combination exists (5X,10I3) for up to 10 levels on each line.

Unspecified levels are assumed unshaded. Shading obstructions are assumed to be infinitely tall as seen by the level considered. Continue for as many lines as necessary to specify the various shading combinations.

Notes: if the entire wall height is unshaded horizontally, input a single blank and continue to step (25). Otherwise input a blank line at the end of the horizontal angle data.

- 25) Ground Reflectance, FORMAT(8F10.3): on a new line, input a ground reflectance coefficient (as a fraction of 1.0) as seen by each level *above ground* (note that the first value corresponds to ground level, not level one necessarily). Provide one value (or blank, default is 0.20) for each level above ground, proceeding from the bottom of the building to the top, eight values per line (8F10.3).

Steel or Concrete Frames

Proceed to step (36) if the structure is loadbearing masonry. Steps (26) to (35) are to be performed for steel or reinforced concrete frames only

- 26) Shelf Angle Location, FORMAT(27I3): on a new line, input either,
- i) the shelf angle spacing (ie., number of levels between shelf angles) followed by the lowest level number at which the veneer is provided (2I3),
 - or ii) the list of level numbers which have shelf angles (or a support) at their bases, proceeding from the bottom to the top of the building. In this case leave the first three spaces of the data line blank (3X,26I3). Default is that no shelf angles are provided.
- 27) Specified Expansion Gap, FORMAT(F10.3,20I3): on a new line, input the specified expansion gap (mm) between the brick and shelf angle followed by the list of levels at which this gap is specified below the floor level shelf angle (F10.3,20I3). Provide a new line for each different gap dimension and its corresponding levels. To indicate the end of the expansion gap data, simply leave the level list blank. If a gap is specified on this line, it becomes the default to which all unspecified gap values are set (if not, the default is 0 mm)
- 28) Shelf Angle Stiffness Parameters, FORMAT(2F10.3): on a new line, input the following two data values;
- i) average shelf angle elastic stiffness in units of kN per mm deflection per m of shelf angle length. Default is $340\ kN/mm/m$.
 - ii) average shelf angle yield force in units of kN per m of shelf angle length. Default is $120\ kN/m$.
- 29) Shelf Angle Corrosion Parameters, FORMAT(2F10.3,I10): on a new line, input the following three data values;
- i) ratio of the volume of iron oxide to the original volume of iron. Default is 2.10 (F10.3)
 - ii) depth of penetration of corrosion from the original shelf angle surface, at the time specified in item (iii), in mm . Default is $1.27\ mm$. (F10.3)
 - iii) shelf angle age, in days since placement, when corrosion has progressed to the depth indicated in item (ii). Default is 3650 days. (I10)

- 30) Wall Cross-section at Column, `FORMAT(8(A2,8X))`: on a new line, input the material names encountered in the wall sandwich at the column location progressing from the outside to the inside. Type each character string starting in the 1st, 11th, 21st, 31st, etc., column of the data line, respectively. The material names recognized by the program are as follows,
- i) BR or BRICK - for clay brick veneer
 - ii) BL or BLOCK - for concrete blockwork
 - iii) CO or CONCRETE - for reinforced concrete column or return wall
 - iv) ST or STEEL - for structural steel column
 - v) BC - for concrete brick veneer (not fully supported)
 - vi) CB - for loadbearing or infill clay blockwork (not fully supported)
 - vii) INS - for insulation
 - viii) AIR - for air cavity (up to two air cavities can be specified in a cross-section)
 - ix) FI or FINISH - for interior wall finish
- 31) Wall Cross-section at Column, cont'd, `FORMAT(12,8X,12)`: on a new line, input the lower and upper level numbers between which, and including, the cross-section specified in step (30) exists at the column location. If this specification includes all building levels (ie., the cross-section is identical at all levels) proceed to step (33). Otherwise complete step (32).
- 32) Wall Cross-section at Column, cont'd, `FORMAT(8(A2,8X))`: on a new line, input the material names (see step (30)) encountered in the wall sandwich at the column location, at levels other than those specified in step (31), progressing from outside to inside.
- 33) Wall Cross-section at Midbay, `FORMAT(8(A2,8X))`: on a new line, input the material names encountered in the wall sandwich at a non-column location, progressing from outside to inside. The material names recognized by the program are the same as in step (30) excluding CO or CONCRETE.
- 34) Wall Cross-section at Midbay, cont'd, `FORMAT(12,8X,12)`: on a new line, input the lower and upper level numbers between which, and including, the cross-section specified in step (33) exists at a non-column location. If this specification includes all building levels, proceed to step (40). Otherwise complete step (35).
- 35) Wall Cross-section at Midbay, cont'd, `FORMAT(8(A2,8X))`: on a new line, input the material names encountered in the wall sandwich at a non-column location, at levels other than those specified in step (34), progressing from outside to inside.

Proceed to step (40).

Loadbearing Masonry Structures

To be completed if the structure is loadbearing masonry only

36) Wall Tie Stiffness Parameters, FORMAT(2F10.3): on a new line, input the following two data values,

- i) assuming that all ties, between the wall midpoints above and below each floor slab, are 'lumped' at the slab location, input the average lumped tie stiffness in units of kN per mm deflection per m width of wall. Default is $1.0 kN/mm/m$ if input value is negative
- ii) input the average vertical force, per m width of wall, required to yield the lumped ties. Default is $0.5 kN/m$ if default is specified in item (i)

37) Wall Cross-section, FORMAT(8(A2,8X)): on a new line, input the material names encountered in the wall sandwich progressing from outside to inside. Type each character string starting in the 1st, 11th, 21st, 31st, etc., column of the data line, respectively. The material names recognized by the program are as follows,

- i) BR or BRICK - for clay brick veneer
- ii) BL or BLOCK - for concrete blockwork
- iii) BC - for concrete brick veneer (not fully supported)
- iv) CB - for clay blockwork (not fully supported)
- v) INS - for insulation
- vi) AIR - for air cavity (up to two air cavities can be specified in a cross-section)
- vii) FI or FINISH - for interior wall finish

38) Wall Cross-section, cont'd, FORMAT(I2,8X,I2): on a new line, input the lower and upper level numbers between which, and including, the cross-section specified in step (37) exists. If this specification includes all building levels, proceed to step (40). Otherwise complete step (39).

39) Wall Cross-section, cont'd, FORMAT(8(A2,8X)): on a new line, input the material names (see step (37)) encountered in the wall sandwich, at levels other than those specified in step (38), progressing from outside to inside.

40) Material Property Specification.

The thermal, dimensional, and deformational properties of each material making up the wall section are read by the program in separate packages, each of which being headed by the appropriate material name (as discussed in steps (30) or (37)). The data following the heading is read according to the requirements of the particular material.

The packages can be placed in any order in the data file, however one package must be provided for each material making up the building wall section (or a blank line as described below). If two air cavities exist in any cross-section, then two air cavity packages must be provided, the first for the outer cavity (and for air cavities in sections with only one air cavity) and the second for the inner air cavity (at sections with two air cavities).

Blanket default values are also supplied for any required but unspecified material packages, which can be accessed by leaving a single blank line following all the specified packages. It is not advisable to rely on the package default option, however, since material dimensions also assume default values. Within each package, default values are provided extensively and are generally obtained by leaving the appropriate data value blank.

In this step, the packages and their data requirements are described individually, as follows.

I) Clay Brick Veneer Data Requirements

- a) Package Name, FORMAT(A2): on a new line, input 'BR' or 'BRICK' starting in the first column.
- b) Tributary Width, FORMAT(F10.3): on a new line, input the total width of the brick veneer measured parallel to the wall face within the tributary area of the supporting element, in m. This value is assumed constant over the entire building height.
- c) Thermal Conductivity, FORMAT(3F10.3): on a new line, input one of the following alternative data sets;
 - i) winter and summer thermal conductivity in $W/m^{\circ}C$ (2F10.3). If the winter conductivity value is blank, it is assumed equal to 70% of the summer conductivity.
 - or ii) clay brick density in kg/m^3 , winter water content, and summer water content in percent by volume (3F10.3). If the winter water content value is blank, default is 1/2 of the summer value. Default density is $2350 kg/m^3$. If this alternative data set is chosen, the summer water content must be specified.
 - or iii) blank line. The summer thermal conductivity defaults to $1.60 W/m^{\circ}C$ and the winter thermal conductivity to $1.10 W/m^{\circ}C$.

d) Heat Flow Parameters, FORMAT(5F10.3) on a new line, input five data values described as follows,

- i) actual brick veneer thickness in mm . Default is 90 mm
- ii) specific heat of the brick veneer in $kJ/kg^{\circ}C$. Default is 0.90 $kJ/kg^{\circ}C$
- iii) total hemispherical emissivity. Default is 0.90.
- iv) solar radiation absorptivity. Default is 0.70
- v) density of the brick veneer in kg/m^3 . Default is the value specified in step (c) or 2350 kg/m^3 (if not specified)

e) Thermal Expansion, FORMAT(3F10.3) on a new line, input three data values described as follows,

- i) ratio of the modular to nominal vertical brick dimensions. Default is 0.851
- ii) coefficient of thermal expansion of either the brick unit or the brick masonry, in the vertical direction, in units of $mm/mm/^{\circ}C$. Note that if the E format is used for data entry, the exponent must be right justified. Default coefficient of thermal expansion of brick masonry is $5.2 \times 10^{-6} mm/mm/^{\circ}C$
- iii) coefficient of thermal expansion of the mortar in $mm/mm/^{\circ}C$. Note that if the masonry coefficient was specified in item (ii) or if the default value is desired, leave this field blank.

f) Moisture Expansion Parameters, FORMAT(I1,9X,2F10.3): on a new line, input up to three data values according to one of the following specifications (depending on data type),

- i) input a 0 (zero) or blank in the first column followed by McDowall and Birtwistle's 4 hour accelerated moisture expansion test value in percent (9X,F10.3). Default is 0.03%.
- or ii) input a 1 in the first column followed by the tested brick unit moisture expansion value, in percent, and the brick age at the time of testing, in days (9X, 2F10.3). Default values are 0.0183% at 500 days. Note that if the default values are used here, the correction factors for humidity and wall orientation are applied.
- or iii) input a 2 in the first column followed by the predicted brick unit moisture expansion value at 20 $^{\circ}C$ and 50% relative humidity, in percent, and the corresponding brick age in days (9X,2F10.3). Default values are 0.0183% at 500 days (adjusted later for humidity and wall orientation).

g) Mortar Shrinkage Parameters, FORMAT(I1,9X,3F10.3): on a new line, input up to four data values according to one of the following specifications (depending on the data type);

- i) input a 0 (zero) or blank in the first column (I1) followed by;
 - 1) estimated ultimate mortar shrinkage in percent. Default is 0.036% (9X,F10.3)

2) air content of mortar in percent. Default is 6.0% (F10.3)

3) sodium chloride to cement ratio in percent. Default is 1.0% (F10.3)

or ii) input a 1 in the first column (I1) followed by the known ultimate mortar shrinkage in percent. Default is 0.036%. (9X,F10.3)

h) Creep Strain Parameters, FORMAT(A3): on a new line, indicate in the first three columns of the data line if the brick was laid 'DRY' or 'WET'. This characteristic reflects on the suction of the brick, as used in Britain. Since their 'wet' brick is probably closer in suction properties to the North American 'dry' brick, the default here is 'WET' and this specification is recommended.

i) Compressive Strength, FORMAT(F10.3,2I5): on a new line, input the brick unit compressive strength in MPa (F10.3), followed by the lower and upper level numbers between which, and including, the specified strength applies (2I5). Proceed from the bottom level to the top with one line for each new strength. If only one strength is present in the building specify it on one line and leave the level limits blank.

j) Elastic Parameters, FORMAT(A2,3X,F10.3,2I5): on a new line, input up to three data values according to one of the following specifications (depending on the data type).

i) in the first two columns (A2) input two letters indicating 'OB'served or 'UN'observed masonry placement, followed by the mortar cement to sand ratio (default is 0.22) (3X,F10.3) and the mortar air content in percent (default is 20%) (I5).

or ii) input the brick masonry elastic modulus in MPa (5X,F10.3) followed by the lower and upper level numbers between which, and including, the specified modulus exists (2I5). Proceed from the bottom level to the top with one line for each new modulus. If only one elastic modulus exists, specify it on one line and leave the level limits blank.

or iii) input a blank line. The elastic modulus of the brick veneer defaults to

$$E_m = 220.6 f'_{br} + 1000 \text{ MPa.}$$

k) Freezing Expansion Parameters, FORMAT(3F10.3): on a new line, input the following three values,

i) the magnitude of the freezing expansion in percent occurring in frozen brick (F10.3). Default is 0%.

ii) the temperature at which the freezing expansion occurs in °C (F10.3). Default is 0 °C.

iii) the magnitude of the permanent expansion in unfrozen brick due to previous freezing, in percent (F10.3). Default is 0%.

II) Concrete Blockwork Data Requirements

- a) Package Name, FORMAT(A2) on a new line, input the characters 'BL' or 'BLOCK' starting in the first column
- b) Tributary Width, FORMAT(F10.3,L2) on a new line, input the total width of the blockwork infill, in metres, within the tributary area of the supporting member (this width is assumed constant wherever blockwork is present) (F10.3) followed by the character 'T' or 'F' (ie, true or false) indicating if a return wall is keyed into the exterior blockwork wall (L2) Default is that no return wall is present (or 'F')
- c) Block Unit Geometry, FORMAT(F10.3,4I5) on a new line, input the following five data values and repeat for as many lines as necessary, working from the bottom level to the top.
 - i) modular block thickness in mm (F10.3) Default is 190 mm
 - ii) net block area over gross block area in percent (I5) Default is 56%
 - iii) percent of cores grouted (I5) Default is 50% if the structure is loadbearing masonry, and 0% otherwise
 - iv) the lower and upper level numbers between which, and including, the preceding parameters apply (2I5) If these values are left blank, the preceding parameters are assumed to be applicable to all levels at which concrete blockwork is provided
- d) Thermal Conductivity, FORMAT(3F10.3,A3,2X,2F10.3) on a new line, input up to four data values according to one of the following specifications (depending on data type),
 - i) summer and winter thermal conductivity of concrete blockwork in $W/m^{\circ}C$ (2F10.3) Under this specification, only one of the two values needs to be specified, the other being estimated on the basis of the change in ambient relative humidity
 - or ii) concrete density in kg/m^3 (no default), three letters indicating 'HOL'low or 'INS'ulated cores (default is 'HOL'), and the summer and winter water content in percent by volume (defaults are calculated from ambient relative humidities) (20X,F10.3,A3,2X,2F10.3).
- e) Specific Heat, FORMAT(3F10.3): on a new line, input up to two data values according to one of the following specifications (depending on data type),
 - i) specific heat of concrete block in $kJ/kg^{\circ}C$ (F10.3)
 - or ii) concrete density in kg/m^3 (default is the value specified in step (d) or $2100 kg/m^3$) followed by the block unit average water content in percent by volume (default is average of values specified in step (d) or is determined from the average annual relative humidity) (10X,2F10.3).
- f) Thermal Parameters, FORMAT(3F10.3): on a new line, input the following three data values,
 - i) total hemispherical emissivity (default is 0.90)

- ii) radiation absorptivity (default is 0.57).
 - iii) coefficient of thermal expansion of concrete blockwork in the vertical direction in $mm/mm/^{\circ}C$ (default is $9.2 \times 10^{-6} mm/mm/^{\circ}C$). Note that if the E format is used to input this value, the exponent term must be right justified
- g) Shrinkage Parameters, **FORMAT(2F10.3,A3,2X,I5)**: on a new line, input up to three data values according to one of the following specifications (depending on data type);
- i) the ultimate block unit shrinkage in percent, the ultimate grout shrinkage in percent (if not present, leave blank), followed by three letters to indicate if the block unit was laid 'WET' or 'DRY' (default is 'WET') (2F10.3,A3).
 - or ii) Three letters to indicate if the block unit was laid 'WET' or 'DRY' (default is 'WET') followed by the average relative humidity at the blockwork location in percent (default is the average annual ambient relative humidity) (20X,A3,2X,I5).
- h) Mortar Shrinkage Parameters, **FORMAT(2F10.3)**: on a new line, input the following two data values,
- i) ultimate mortar shrinkage in percent (default is the mortar shrinkage value specified for the brick veneer, if already encountered, or is estimated from Equation (3.77))
 - ii) ratio of the modular to nominal vertical block dimensions (default is 0.95).
- i) Elastic Parameters, **FORMAT(A3)**: on a new line, input one of the following two or three letter control codes, starting in the first column;
- i) 'EM' or blank, to indicate that the values specified in step (j) are the blockwork elastic moduli.
 - or ii) 'FM1' to indicate that compressive strengths will be specified in step (j) and that the elastic modulus of the ungrouted blockwork is to be determined by a combination of unit elastic moduli (Eq. (3.84)).
 - or iii) 'FM2' to indicate that compressive strengths will be specified in step (j) and that the elastic modulus of the ungrouted blockwork is to be determined using Equation (3.81)
- j) Elastic Parameters, cont'd, **FORMAT(variable)**: on a new line, input data values according to one of the following specifications, depending on the control code specified in the previous line;
- i) for 'EM' or blank: **FORMAT(F10.3,2I5)** input the elastic modulus of the blockwork in MPa, followed by the lower and upper level numbers between which, and including, the specified elastic modulus is applicable. Default elastic modulus is 9170 MPa. Continue for as many lines as necessary, working from the bottom to the top level. If the level limits are left blank, the specified (or default) elastic modulus is assumed applicable to all levels where blockwork is provided.

- or ii) for 'FM1' or 'FM2' FORMAT(2F10.3,2I5) input the net area block unit compressive strength and the mortar compressive strength, in *MPa*, followed by the lower and upper level numbers between which, and including, these values are applicable (default block unit and mortar compressive strengths are 13.5 and 12.0 *MPa*, respectively). Continue for as many lines as necessary, working from the bottom to the top level. If the level limits are left blank, the specified (or default) strengths are assumed applicable to all levels where the blockwork is provided.
- k) Grout and Reinforcement Parameters, FORMAT(3F10.3,2I5): if the blockwork is ungrouted, proceed directly to step (m). Otherwise, on a new line, input the following five data values,
- i) grout compressive strength at 28 days in *MPa* (default is 11.0 *MPa*)
 - ii) area of vertical reinforcing bar, or group of bars, in mm^2 (default is 0 mm^2)
 - iii) horizontal spacing of the reinforcing bar or group specified in item (ii), in *mm*. If this spacing is left blank, the steel area specified in item (ii) is assumed to be the total steel area provided within the blockwork tributary length
 - iv) lower and upper level numbers between which, and including, the specified grout and reinforcement parameters are applicable. Continue for as many lines as necessary, working from the bottom to the top level. If the level limits are left blank, the specified (or default) values are assumed applicable to all levels where blockwork is provided.
- l) Reinforcement Parameters, FORMAT(2F10.3): if the blockwork is unreinforced, proceed directly to step (m). Otherwise, on a new line, input the following two data values;
- i) modulus of elasticity of the blockwork reinforcement in *MPa* (default is 200 000 *MPa*).
 - ii) coefficient of thermal expansion of the blockwork reinforcement in $\text{mm/mm}/^\circ\text{C}$ (default is $12.06 \times 10^{-6} \text{ mm/mm}/^\circ\text{C}$). Note that if the E format is used to input this value, the exponent term must be right justified.
- m) Blockwork - Slab Gap, FORMAT(8F10.3): if the structure is loadbearing masonry, proceed to the next material package specification. Otherwise, on a new line, specify the gap left between the blockwork panel and the upper slab at each level, working from level 1 to the roof. Provide 8 values per line and continue for as many lines as necessary (ie., 15 levels will require 2 lines). The default gap at any level is assumed to be equal to the next lower specified value or 0 *mm* if lower levels are left unspecified.

III) Interior Wall Finish Data Requirements

- a) Package Name, FORMAT(A2): on a new line, input the letters 'FI' or 'FINISH' starting in the first column.
- b) Thermal Parameters, FORMAT(I5,3F10.3,2F5.2,2F10.3): on a new line, input the following 8 data values;
 - i) finish type code (I5) having one of the following values;
 - 1 if the variation in thermal conductivity with temperature ($^{\circ}K$) is known and specified, along with specific heat and density, later in this line
 - 0 or blank if the summer and winter thermal conductivities, specific heat, and density are specified later in this line
 - 1 if the wall finish is gypsum board
 - 2 if the wall finish is wood fibreboard
 - 3 if the wall finish is mineral aggregate board
 - 4 if the wall finish is softwood plywood, lumber, or mat-formed particle board
 - 5 if the wall finish is stucco
 - ii) density of the wall finish in kg/m^3 (F10.3). This specification is optional if the type code is greater than zero but takes precedence over the approximate default value.
 - iii) thickness of the wall finish in mm (default is 12.0 mm) (F10.3)
 - iv) specific heat of the wall finish in $kJ/kg^{\circ}C$ (F10.3). This specification is optional if the type code is greater than zero but takes precedence over the approximate default value.
 - v) total hemispherical emissivity of the wall finish (default is 0.90) (F5.2)
 - vi) radiation absorptivity of the wall finish (default is 0.60) (F5.2)
 - vii) for a type code of -1, input the coefficients A and B from which the thermal conductivity can be determined internally as;

$$k = A + B(T) \quad (W/m^{\circ}C)$$

where T is the wall finish temperature in $^{\circ}K$.

for a type code of 0 (zero) or blank, input the summer and winter thermal conductivities of the wall finish in $W/m^{\circ}C$. If the type code is greater than zero, the thermal conductivities are generated internally. (2F10.3)

IV) Air Cavity Data Requirements

- a) Package Name, FORMAT(A2): on a new line, input the letters 'AIR' or 'AI' starting in the first column.
- b) Air Cavity Thickness, FORMAT(F10.3): on a new line, input the air cavity thickness in mm (default is 25 mm).

Note: the program allows for up to two air cavities in the same wall section, therefore this package can be repeated. The first occurrence is assumed to apply to the outer air cavity (and to sections with just one air cavity) and the second to the inner air cavity.

V) Insulation Data Requirements

- a) Package Name, FORMAT(A2): on a new line, input the letters 'INS' starting in the first column
- b) Thermal Parameters, FORMAT(I5,3F10.3,2F5.2,2F10.3): on a new line, input the following 8 data values,
 - i) insulation type code number having one of the values shown in Table E.1. (I5)
 - ii) density of the insulation in kg/m^3 (F10.3). This specification is optional if the type code is greater than zero but takes precedence over the approximate default value.
 - iii) thickness of the insulation in mm (F10.3). Default is 25 mm.
 - iv) specific heat of the insulation in $\text{kJ/kg}^\circ\text{C}$ (F10.3). This specification is optional if the type code is greater than zero but takes precedence over the approximate default value.
 - v) total hemispherical emissivity of the insulation (F5.2). Default is 0.90.
 - vi) radiation absorptivity of the insulation (F5.2). Default is 0.60.
 - vii) for a type code of -1, input the coefficients A and B from which the thermal conductivity of the insulation can be determined internally as;

$$k = A + B(T) \quad (\text{W/m}^\circ\text{C})$$

where T is the insulation temperature in $^\circ\text{K}$.

For a type code of 0 or blank, input the summer and winter thermal conductivities of the insulation in $\text{W/m}^\circ\text{C}$. If the type code is greater than 0, the thermal conductivities are generated internally. (2F10.3)

Table E.1 Insulation type code numbers.

Insulation Type Code	Description
-1	coefficients for the conductivity equation $k = A + B(T)$, specific heat, and density are specified
0	summer and winter thermal conductivities, specific heat, and density are specified
1	Laminated asbestos felt
2	146 laminations/m
3	73 laminations/m
4	Corrugated asbestos
5	Rock wool (from Eckert, 31)
6	Mineral fibre blankets and felts
7	Cellular glass
8	Glass fibreboard
9	Rigid polystyrene - extruded
10	- molded beads
11	Polyurethane
	Loose fill
12	Silica aerogel
13	Vermiculite ($64 < \rho < 96 \text{ kg/m}^3$)
14	Perlite
15	Vermiculite ($110 < \rho < 130 \text{ kg/m}^3$)
16	Fibreglass batts
17	Preformed fibreglass
18	Glass wool
19	Rock wool (from General Electric, 37)
20	Styrofoam
21	Gypsum board
22	Wood fibreboard

VI) Steel Column Data Requirements

- a) Package Name, FORMAT(A2): on a new line, input the letters 'ST' or 'STEEL' starting in the first column
- b) Cross-sectional Area, FORMAT(8F10.3): on a new line, input the steel column cross-sectional area at each level starting from level 1 and proceeding to the roof. Provide one value for each level and continue for as many lines as necessary (ie., 15 levels will require 2 lines) with 8 values per line. Default is the next lower specified value
- c) Steel Properties, FORMAT(2F10.3): on a new line, input the steel elastic modulus in MPa (default is 200 000 MPa) followed by its coefficient of thermal expansion in $mm/mm/^{\circ}C$ (default is $12.06 \times 10^{-6} mm/mm/^{\circ}C$)

VII) Concrete Column Data Requirements

- a) Package Name, FORMAT(A2): on a new line, input the letters 'CO' or 'CONCRETE' starting in the first column
- b) Thermal Parameters, FORMAT(5F10.3): on a new line, input three data values according to one of the following specifications (depending on the data type);
 - i) summer and winter thermal conductivity of the concrete in $W/m^{\circ}C$ and the concrete density in kg/m^3 (2F10.3, 20X, F10.3). If only one of the thermal conductivity values is specified, the other is determined internally. The default density is 2300 kg/m^3
 - or ii) summer and winter free water content, in percent by volume, and the concrete density in kg/m^3 (20X, 3F10.3). The summer and winter water contents default to 3% and 1.5%, respectively, and the density to 2300 kg/m^3 .
- c) Thermal Properties, FORMAT(4F10.3): on a new line, input the following four data values;
 - i) specific heat of the concrete in $kJ/kg^{\circ}C$ (default is 0.94 $kJ/kg^{\circ}C$).
 - ii) summer water content in percent by volume if the specific heat specified is that of dry concrete (otherwise leave blank).
 - iii) total hemispherical emissivity (default is 0.90).
 - iv) radiation absorptivity (default is 0.57).
- d) Coefficient of Thermal Expansion, FORMAT(15,F10.3,7F5.3): on a new line, input up to 8 data values according to one of the following specifications (depending on data type);
 - i) interior average relative humidity in percent (default is the average annual ambient relative humidity) followed by the coefficient of thermal expansion of concrete in $\mu m/m^{\circ}C$ (note the units) (default is 9.51 $\mu m/m^{\circ}C$) (15,F10.3).

or ii) interior average annual humidity in percent (default is the average annual relative humidity) followed by;

- 1) coef. of thermal exp. of the coarse aggregate in $\mu m/m^{\circ}C$
- 2) coef. of thermal exp. of the fine aggregate in $\mu m/m^{\circ}C$
- 3) coef. of thermal exp. of the cement paste in $\mu m/m^{\circ}C$
- 4) proportion by volume of coarse aggregate (as a fraction of 1.0)
- 5) proportion by volume of fine aggregate
- 6) proportion by volume of cement paste
- 7) environmental condition coefficient (1.0 if airconditioned, 0.86 if open to the exterior)

Default values are supplied for all of the preceding items (15,10X,7F5.3)

- e) Concrete Shrinkage, FORMAT(F10.3): on a new line, input the unadjusted ultimate shrinkage of the concrete in percent, as determined (or estimated) from concrete test cylinders stored at 20 $^{\circ}C$ and 40% relative humidity. Default is estimated from Eq. (3.17).
- f) Column Geometry, FORMAT(2F10.3,2I5): on a new line, input the following four data values,
 - i) column thickness measured in mm perpendicular to the wall face (F10.3)
 - ii) column width measured in mm parallel to the wall face (F10.3)
 - iii) lower and upper level numbers between which, and including, the above parameters apply (2I5). Proceed from level 1 to the roof for as many lines as necessary. If the level limits are left blank, the column dimensions are assumed constant throughout the building elevation.
- g) Elastic Parameters, FORMAT(3F10.3,2I5): on a new line, input the following four data values,
 - i) either the 28 day modulus of elasticity of the concrete, in MPa, in the first 10 spaces (F10.3), or the 28 day compressive strength, in MPa in the second 10 spaces (10X,F10.3).
 - ii) either the vertical reinforcement steel area, in mm^2 , or the reinforcement ratio ρ in the third 10 spaces (F10.3). If the specified value is less than 0.09, it is assumed to be the reinforcement ratio.
 - iii) lower and upper level numbers between which, and including, the previous specifications apply (2I5). Continue for as many lines as necessary working from level 1 to the roof. If the level limits are left blank, the specified values are assumed to apply at all levels.
- h) Reinforcement Properties, FORMAT(2F10.3): on a new line, input the following two data values;

- i) coefficient of thermal expansion of the reinforcement in $mm/mm/^\circ C$ (default is $12.06 \times 10^{-6} mm/mm/^\circ C$) (F10.3)
- ii) modulus of elasticity of the reinforcement in MPa (default is $200\,000 MPa$) (F10.3)

- 41) Timing Parameters, FORMAT(10I8) on a new line, input up to 10 dates, appearing in the form of right justified 8 digit integers (DAY:MONTH YEAR or DDMMYY, for instance, Feb 8, 1957 would appear as 080257), as follows,
- i) date construction of first level (lowest basement) was begun (I8)
 - ii) date construction of roof was completed (it is assumed that the building became air-tight at this time) (I8)
 - iii) up to 8 dates at which time the exterior wall movements and stresses are desired (ie. at 2, 5, 10 years, etc) (8I8)
- 42) Timing Form, FORMAT(A2) on a new line, input one of the following control words, starting in the first column,
- i) 'EX' or 'EXPLICIT' to indicate that the dates of creation and/or placement of each wall element at each level will be individually specified in the following data lines
 - ii) 'AP' or 'APPROXIMATE' to indicate that the times of creation and/or placement of each wall element at each level will be determined approximately from the following single data line This is the default condition

If the timing form is EXPLICIT, proceed to step (47) If not, complete steps (43) and (44) for concrete and steel frame structures or steps (45) and (46) for loadbearing masonry structures

Concrete and Steel Frame Structures

- 43) Approximate Timing, FORMAT(7I5) on a new line, input the following 7 data values (no defaults provided),
- i) the time in days required to construct one complete level
 - ii) the level that frame construction had started on when the blockwork was first laid at its starting level
 - iii) the level that frame construction had started on when the brick veneer was first laid at its starting level
 - iv) the time lapse, in days, between the completion of a brick panel and placement of the overlying shelf angle
 - v) the average kiln-to-wall time lapse, in days, for brick units at any level
 - vi) the average factory-to-wall time lapse, in days, for block units at any level
 - vii) the average time lapse, in days, between creation of a column at a level and the removal of the shoring at the same level
- 44) Approximate Timing, cont'd, FORMAT(2I5): if both the brick veneer and the blockwork are continuous over their respective heights (ie., provided at all levels between their lowest

and highest levels of occurrence), then this line is ignored and the data file is complete. If a break of a level or more does occur in either the brick veneer or the blockwork, then provide the following two data values on a new line to conclude the data file;

- i) the level that the frame construction had started on when the blockwork was first laid at its upper, or second, starting level
- ii) the level that the frame construction had started on when the brick veneer was first laid at its upper, or second, starting level

Loadbearing Masonry Structures

45) Approximate Timing, FORMAT(5I5) on a new line, input the following 5 data values (no defaults provided),

- i) the time in days required to construct one complete level
- ii) the average time lapse, in days, between completion of the loadbearing blockwork wall and the placement of the overlying slab
- iii) the level that the frame construction had started on when the brick veneer was first laid at its starting level
- iv) the average kiln-to-wall time lapse, in days, for brick units at any level
- v) the average factory-to-wall time lapse, in days, for block units at any level

46) Approximate Timing, cont'd, FORMAT(I5): if the brick veneer is continuous over its height (ie, provided at all levels between its lowest and highest levels of occurrence), then this line is ignored and the data file is complete. If a break of a level or more does occur in the brick veneer, then provide the following data value on a new line to conclude the data file;

- i) the level that the frame construction had started on when the brick veneer was first laid at its upper, or second, starting level

47)

Explicit Timing Data Requirements

If a data value is missing (blank) in the following explicit timing data lines, its value is internally estimated by calculating the time increment between the preceding two data values and adding this increment to the preceding data value. This implies that at least the first two time values for any category or step must be specified. If the blockwork or brick veneer are discontinuous over their heights (ie., missing at intermediate levels), the first two time values corresponding to the first two levels above the 'break' must also be specified for the corresponding brick or block categories only. For example, if brick veneer is provided at all levels of a 15 storey structure except for the 10th level, then at least the first, second, tenth, and eleventh brick production and placement time values must be specified (note that the 10th value corresponds to the 10th level of brick veneer provided, which occurs at the 11th building level).

Complete steps (48) to (54), if the building is a reinforced concrete or steel frame structure, or steps (55) to (59), if the building is a loadbearing masonry structure.

Concrete or Steel Frame Structure

- 48) Time of Column Placement, FORMAT(10I8): on a new line, specify the date of completion of the column placement at each level (DDMMYY) starting from level 1 and working up. Provide 1 value for each level, 10 values per line, and continue for as many lines as necessary (ie., a 25 storey structure will require 3 lines even if they are largely blank).
- 49) Shoring Removal Timing, FORMAT(10I8): on a new line, specify the date that the shoring was removed from each level (DDMMYY) starting from level 1 and working up. Provide 1 value for each level, 10 values per line, and continue for as many lines as necessary.
- 50) Blockwork Placement Timing, FORMAT(10I8): on a new line, specify the date (DDMMYY) that the blockwork panel was completed at each level. Provide 1 value for each level that the blockwork is provided, working upwards, 10 values per line, and continue for as many lines as necessary (ie., a 25 storey structure with only 10 levels of blockwork will require only 2 lines).
- 51) Block Unit Production Timing, FORMAT(10I8): on a new line, specify the date (DDMMYY) that the block units at each level were produced at the factory. Working upwards, provide 1 value for each level that the blockwork is provided, 10 values per line, and continue for as many lines as necessary.
- 52) Brick Veneer Placement Timing, FORMAT(10I8): on a new line, specify the date (DDMMYY) that the brick veneer panel at each level was completed. Working upwards, provide 1 value for each level that the brick veneer is provided, 10 values per line, and continue for as many lines as necessary.
- 53) Brick Unit Production Timing, FORMAT(10I8): on a new line, specify the date (DDMMYY) that the brick units used at each level were produced. Working upwards, provide 1 value

for each level that the brick veneer is provided, 10 values per line, and continue for as many lines as necessary.

- 54) Shelf Angle Placement Timing, FORMAT(10I8): on a new line, specify the date (DDMMYY) that the shelf angle at the top of a level was placed. Working upwards, provide one value for each shelf angle provided above the foundation, 10 values per line, and continue for as many lines as necessary.

This completes the data file.

Loadbearing Masonry Structures

- 55) Blockwork Placement Timing, FORMAT(10I8): on a new line, specify the date (DDMMYY) that the blockwork wall at each level was completed. Start from level 1 and work upwards, providing 10 values per line. Continue for as many lines as necessary (ie, a 25 storey structure will require 3 lines even if they are largely blank).
- 56) Slab Placement Timing, FORMAT(10I8): on a new line, specify the date (DDMMYY) that the overlying slab was placed at each level. Start from level 1 and work upwards, providing 10 values per line. Continue for as many lines as necessary.
- 57) Block Unit Production Timing, FORMAT(10I8): on a new line, specify the date (DDMMYY) that the block units used at each level were produced. Start from level 1 and work upwards, providing 10 values per line. Continue for as many lines as necessary.
- 58) Brick Veneer Placement Timing, FORMAT(10I8): on a new line, specify the date (DDMMYY) that the brick veneer panel at each level was completed. Working upwards, provide 1 value for each level that the brick veneer is provided, 10 values per line, and continue for as many lines as necessary (ie., a 22 storey structure with only 19 levels of brick veneer will require only 2 lines).
- 59) Brick Unit Production Timing, FORMAT(10I8): on a new line, specify the date (DDMMYY) that the brick units used at each level were produced. Working upwards, provide 1 value for each level that the brick veneer is provided, 10 values per line, and continue for as many lines as necessary.

This completes the data file.

Running the Program

At this point, the user will have a data file which, for the purposes of discussion, will be assumed to be called BLDG. Because a program the size of VENEER is usually required to run at off-peak hours, it is generally best to run it in batch mode. The batch program required to run VENEER would have the following contents,

```
!JOB number,name,9,LOCAL
!LIMIT (TIME,5),(CORE,50),(LO,80)
!SET F 2/BLDG,IN
!SET F 3 LP
!RVENEER
```

If the file listed above is called XEQVENEER, the user simply has to type,

```
!BATCH XEQVENEER
```

to run the program. Also provided is a much smaller version of RVENEER (the run module), called READBLD, which does nothing except read the data from BLDG and print the desired building information (specified in step (2) of the data input procedure). This allows the user to check his data file without waiting until off-peak hours. The batch file (XRDR) to run READBLD would appear as follows;

```
!JOB name,number,9,LOCAL
!LIMIT (TIME,2),(CORE,40),(LO,50)
!SET F:2/BLDG;IN
!SET F:3 LP
!READBLD
```

Note that in both cases, the output is printed at the Administration building at Carleton University (LOCAL). To run this smaller version, simply type;

```
!BATCH XRDR
```

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